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General Investigation of Lake Andes, South Dakota

Lake-Level Frequency Analysis Report

March 2003

US Army Corps of Engineers ®
Omaha District

GENERAL INVESTIGATION OF LAKE ANDES, SOUTH DAKOTA

LAKE-LEVEL FREQUENCY ANALYSIS REPORT

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U.S. Army Corps of Engineers
Omaha District
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Omaha, NE 68102-1618

EXECUTIVE SUMMARY

A general investigation was completed in cooperation with the U.S. Fish and Wildlife Service (USFWS) to determine existing and future lake-level frequencies of Lake Andes in south-central South Dakota. Lake Andes is a natural glacial lake, similar in nature to the Waubay Lakes in South Dakota and Devil's Lake in North Dakota, and it is influenced significantly by plains snowmelt, rainfall runoff, and evaporation. Deterioration of its water control structures has rendered it difficult for USFWS to manage the lake for waterfowl production. As a result USFWS is seeking to improve existing structures and raise embankments for better lake management.

A hydrologic model was created using the Wetland Hydrologic Analysis Model (WHAM). WHAM is a daily time-step water budget model developed by USACE Omaha District, which accounts for daily precipitation, evaporation, seepage, runoff, outflow, and wetland storage change. The model was calibrated to historic records from 1960 to 2002 with considerations for hydrologic trends in the glaciated watershed, changes to water control structures, and operational adjustments. The lake was initially simulated as a single water surface. Then the simulation was extended to a model with three interdependent lake units separated by dike/road embankments and connected with water control structures. From the calibrated model, 100 years of lake-level record were simulated under existing and future modified conditions, which included lowered outlet crest elevations. Lake-level frequency curves were developed from the simulated peak lake levels then combined with wind-wave analysis runup to provide USFWS with lake levee and outlet design recommendations.

Maximum coincident lake levels occurring at the 10%(10-year), 2%(50-year), and 1%(100-year) frequencies are 1439.70, 1441.10, and 1442.00 ft MSL, respectively. Existing road elevations reported in NGVD 1929 are 1441.05 ft MSL on the North Dike, and 1439.25 ft MSL on the South Dike, or 1440.25 ft MSL if the raised bridge deck elevation is used on the South Dike. Static pool elevations at the 1% frequency will overtop the dikes while 2% frequency elevations may partially overtop the dikes. The lake level frequency and coincident frequency results showed limited differences in lake levels at prescribed frequencies between the two methods. This limited difference may indicate that starting lake-levels marginally influence peak lake-levels throughout the year.

Future modified conditions of the Lake Andes outlet at the U.S. 281 box culverts could include a 2.0-ft lowering of the weir inlet elevation to 1435.25 ft MSL, or a 4.0-ft lowering of the weir inlet elevation to 1433.25 ft MSL (complete stoplog removal). Operating Lake Andes at lower levels would substantially affect the lake-level frequencies. Maximum lake levels that could occur under subsurface conduit outlet control for the 2.0-ft lower crest elevation at the 10%(10-year), 2%(50-year), and 1%(100-year) frequencies are 1437.15, 1440.20, and 1441.25 ft MSL, respectively. Maximum lake levels that could occur under the 4.0-ft lower crest elevation at the 10%(10-year), 2%(50-year), and 1%(100-year) frequencies are 1436.55, 1439.60, and 1440.95 ft MSL, respectively.

Wave setup, as well as run-up, will cause lake levels to push substantially above static pool elevations during windstorms. Wave action can damage or destroy unprotected earthen embankments. The largest waves for the road embankments are found at the North Dike. A 10-year windstorm on a pool at elevation 1437.25 will nearly wash over the road with a wave projection to 1441.0 feet. A similar storm projects waves to 1440.2 on the South Dike. Waves will wash over the current road embankment, which is at elevation 1439.25.

A 50-year flood will overtop the North Dike for roughly a day by a maximum of 0.35 feet to elevation 1441.40 ft MSL when the starting pool is at elevation 1435.8 feet MSL. Higher starting pools result in longer and higher overtopping situations. In the highly unusual situation where the entire Lake Andes system is high (pool elevation of 1441 feet and above), the road would be overtopped for a longer time, but the overflow velocity would be lower given the backwater conditions from the outlet at Highway 281.

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1. INTRODUCTION

Lake Andes is a natural glacial lake located in south-central South Dakota. The U.S. Fish and Wildlife Service (USFWS) manages it as part of the Lake Andes National Wildlife Refuge. Figure 1.1 in this section notes the location of Lake Andes in relation to other prominent locations within the vicinity. Since Lake Andes is a natural lake, its water levels and volumes depend greatly on the fine balance of rainfall runoff, snowmelt runoff, and lake evaporation, and lake outflows. Lake dikes and outlet structures constructed in the 1940s and 1960s enable USFWS to manage lake levels for desired depths and elevations; however, reduced outflow capacities of these structures have presented some difficulties in achieving these desired levels. In addition, historically wet climatological cycles have resulted in localized inundation of lake levee roads, and dry cycles have led to periods of very low lake levels. Lake-level frequency analyses were performed on simulated lake levels under various conditions to provide design information for road and outflow structure improvements.

1.1 History

Lake Andes became state property in 1889 when South Dakota received its statehood. With the only natural outlet discharging water from Johnson Bay to Choteau Creek to the East, Lake Andes experienced high lake levels and developed a reputation as a prime bass fishing lake in the area during the 1920s. Flooding of agricultural ground during this time period prompted farmers to push for Congressional establishment of a high water level limiting water levels to 1437.25 ft above Mean Sea Level (ft MSL). In 1934 an outlet tube was constructed at the south end, releasing water to the Missouri River. A severe drought that followed in the 1930s resulting in very low lake levels again prompted better management of water within the lake. In the 1930s, water from Garden Creek was diverted to Lake Andes, and in 1936 the Owen's Bay dike and outlet structure were constructed. Providing more water and managing Owen's Bay separately allowed USFWS to manage the lake for migratory waterfowl and other wildlife. In 1942 construction of the North and South Dikes was completed dividing the lake into three lake units as it is today. The purpose of the division was to concentrate water in individual units during droughts to provide for improved migratory bird habitat. One resulting problem of the lake sub-division was the reduction of storage in the North Pool coupled with the limited outflow capacity. High lake levels in the North Pool in the 1960s again prompted construction of an emergency spillway in the North Dike to increase the dike's outflow capacity and reduce flooding of agricultural land in the North Pool Basin. Recent lake levels have experienced fewer extremes; however, wet periods during the mid-1980s, 1990s, and early 2000s caused high lake levels and submergence of the roads three times since 1980. Recent flooding and a decrease in the desired level of lake management are partly due to reduced outflow capacities caused by sediment deposition and structure deterioration.

1.2 Purpose of Study

Given recent dike road submergence and lake management capabilities, USFWS has communicated several goals regarding lake management and pool levels. First, USFWS would like to upgrade dike roads for safe travel during most conditions, specifically when a 50-year return period rainfall event is imposed upon a selected water surface. Secondly, USFWS would like to be able to rid the lake of "rough" fish species by drawing the lake down when needed. A shallower lake with more associated wetlands would also provide a more suitable environment for waterfowl production. Third, USFWS would like to improve and expand existing lake unit outflow structures for more efficient lake-level management. Currently, the condition of the Lake Andes outlet structure, outlet channel, and the capacity of the subsurface conduit draining water to Garden Creek greatly limit the ability of USFWS to efficiently manage lake levels. USFWS has expressed interest in managing lake levels 2.0 to 4.0 feet lower than current levels under certain circumstances. In addition, they wish to increase outflow capacities by improving the outflow system, which would allow them to flush water from the lake more rapidly when necessary.

The purpose of this study was to develop lake-level frequency relationships for Lake Andes, South Dakota. The Denver and Lake Andes Refuge Offices of the Fish and Wildlife Service furnished much of the data used in the study. Additional information was obtained during site reconnaissance in late October 2002. Using the data, a daily time-step hydrologic model was developed to simulate basin runoff and lake conditions. The model was calibrated to observed lake elevation data and was used to simulate a continuous period of record from 1903 – 2002. A frequency analysis was performed on the simulated annual maximum lake levels to determine the probability of exceeding various lake elevations. A wind and wave analysis was performed to determine the effects of wind setup and wave run up. Scenarios, using various combinations of pool elevations and windstorms, were developed in order to forecast design elevations for the road embankments and related erosion protection. Ten and 50-year peak flood inflows were routed through the North Unit pool in order to provide design information for sizing the bridge opening under the reconstructed road.

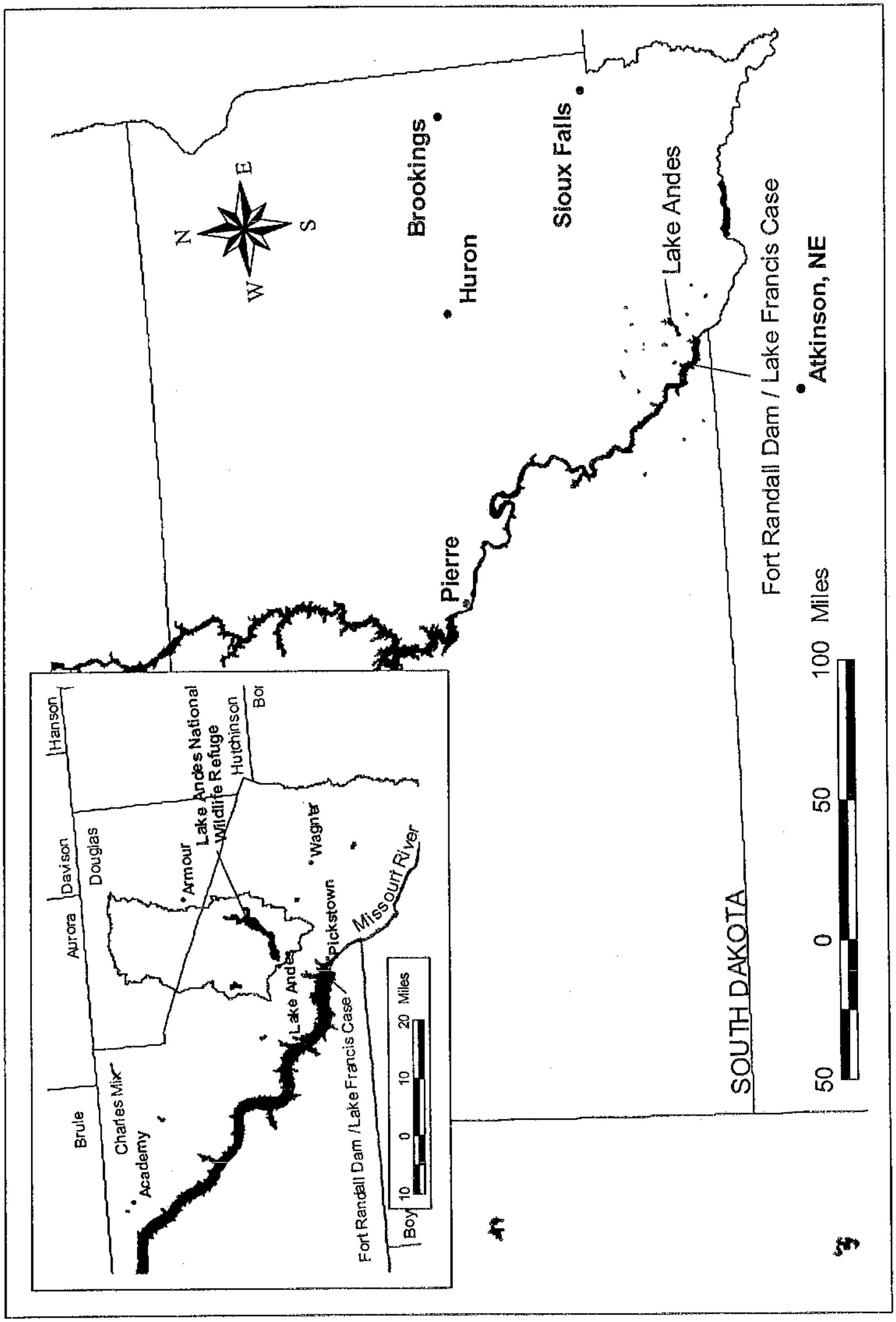

Figure 1.1 Lake Andes study site and data source locations.

2. MODEL CONSTRUCTION

Model input information was prepared for the Wetland Hydrologic Analysis Model Version 3.4 (WHAM) for the simulation of Lake Andes. Daily precipitation, minimum and maximum temperature, and evaporation were compiled from local weather stations and supplemented with information from additional weather stations in the region. Delineation of the model into four lake units using GIS supplemented with local information allowed the development of the Lake Andes model from both a combined and divided watershed approach. Lake elevation area-capacity curves and outflow rating curves were constructed with information provided by USFWS. Finally, monthly hydrologic parameters were estimated using available regional data adopted from other hydrologic studies.

2.1 Model Description

The Wetland Hydrologic Analysis Model V 3.4 (WHAM), was developed by the Omaha District of the Corps of Engineers to allow the continuous simulation of a lake and its adjacent watershed. WHAM is a daily time-step water budget model that accounts for daily precipitation, evaporation, seepage, runoff, outflow, and wetland storage change. WHAM uses a simple water budget principle (inflow = outflow + change in storage) to determine wetland or lake water surface elevations on a daily basis. WHAM requires inputs of daily precipitation, temperature and evaporation. WHAM simulates watershed runoff using a simplified soil moisture index curve for determining excess precipitation and a daily time-step unit hydrograph. Additional inflows from gaged streams, pumps or diversions can also be specified. With the input of daily minimum and maximum temperature, WHAM may also simulate snow accumulation and snowmelt runoff. Finally, WHAM determines outflow from the wetland or lake by an elevation-storage-outflow curve, with the capability to adjust outflow for tailwater conditions.

2.2 Precipitation and Temperature Data

Data from Pickstown, SD were used for model input for the years 1952 through 2002. Where information from the Pickstown weather station data was missing during that period, it was supplemented with data from Wagner, SD. Locations of precipitation and temperature data are shown on the maps in either Figure 1.1 or Figure 2.1.

In order to obtain meteorological data more representative of the Lake Andes area, a survey was made of the available digital data closer to Pickstown. Most of the precipitation and temperature data for the period 1902 – 1952 were obtained from measurements taken at Armour, SD located 20 miles northeast of Pickstown and near the upper end of the Andes Creek watershed. A considerable amount of data was missing for Armour between 1909 and 1916, with sporadic data missing in other years. The data missing from the Armour weather station were filled in using data from Academy, SD, which is located about 38 miles northwest of Pickstown. After filling in the missing data using records from Academy, there were only a scattered few days (less than 20) of missing data remaining. Those few days were filled in using the records from Atkinson, NE, which is located 44 miles to the southwest of Pickstown.

2.3 Evaporation Data

Monthly evaporation data for Pickstown were available from the NCDC Hydrodata Databases available on CD-ROM for the period 1951 – 1998. Data for the years 1999 through 2002 were obtained from daily pan evaporation readings posted on monthly weather data sheets compiled by the Ft. Randall Dam/Lake Francis Case Project at Pickstown, SD. Pan evaporation was not recorded during the ice-affected months of November through March; therefore, average monthly pan evaporation was estimated for these months using an assumed annual distribution. The annual distribution of evaporation was derived using monthly evaporation derived using the Penman Equation for Huron and Sioux Falls, SD, which was provided in Table II of NWS 34. The annual distribution and April to October evaporation fractions were applied to the annual pan evaporation estimates provided in NWS 33 to derive all monthly pan evaporation values from 1951 through 2002.

Data for the period 1902 – 1951 were obtained from monthly free water surface estimates prepared by South Dakota State Climatologist Al Bender for the Brookings area. The free water surface data were converted into pan evaporation data using the pan coefficient 0.75 at Brookings. The Brookings evaporation data was used to estimate the Pickstown evaporation data by multiplying the Brookings data by a factor of 1.1 to reflect the greater evaporation and the lower pan coefficient at Pickstown. This factor was obtained by consulting "Map 3 of 4: Annual FWS Evaporation" from National Weather Service Technical Report NWS 33. On that map, Brookings was shown to have an average annual free water surface evaporation of 41 inches while Pickstown was shown to have an evaporation of 45 inches. The ratio of the Pickstown to Brookings values was approximately 1.1. Locations of Brookings, Huron, and Sioux Falls, SD, are shown on the map in Figure 1.1.

2.4 Supplemental Hydrologic Data

The U. S. Geological Survey obtained spot lake level readings on Lake Andes and current meter measurements on Andes Creek and its tributaries since the late 1980s as part of a long-term water quality-monitoring program. A summary of the USGS water quality monitoring sites is provided in Table 2.1. There are no recording gages in the program, therefore all data are provided as spot measurements in the Water Resources Data publications.

Most of the data provided consists of water quality parameters, but discharge, stage and lake depth were taken at each site. Additional lake elevation readings were estimated for the North Unit, using the gage heights recorded from the staff gage at the "Lake Andes above Ravinia, SD" site. This was done to estimate additional data points for years when there were few measurements, such as in 1999.

Table 2.1 USGS Water Quality Measurement Sites in the Lake Andes Watershed

STATION NAME	NUMBER	SEC	T (N)	R (W)	PARAMETER	SINCE
Andes Creek nr. Armour, SD	6452380	3	97	64	Discharge	1983
Lake Andes Trib. #1 nr L. Andes, SD	6452389	25	97	65	Discharge	1984
Lake Andes Trib. #2 nr L. Andes, SD	6452386	18	97	64	Discharge	1984
Lake Andes Trib. #3 nr Armour, SD	6452383	5	97	64	Discharge	1986
Lake Andes above Ravinia, SD	6452390	16	97	64	Gage Height	1990
Lake Andes nr. Ravinia, SD	6452391	29	97	64	Depth	1990
Owens Bay nr. Ravinia, SD	6452403	5	96	64	Depth	1990
Lake Andes above. L. Andes, SD	6452406	1	96	65	Depth	1990

2.5 Lake Area/Capacity Tables

The U.S. Fish and Wildlife Service Lake Andes Refuge office provided elevation-area/capacity tables for the North, Middle and South Units and Owens Bay. Areas and storage capacities were determined from Solberg's 1945 contour map. Curves were constructed from the tables and extrapolated to elevation 1444.0 ft above mean sea level (MSL) for the WHAM model and are provided in the Appendix as Figures A.1 through A.5. Both individual unit storage tables and a combined Lake Andes table, including all four units, were constructed at 0.5 to 1.0 foot elevation intervals for the model.

2.6 Contributing Drainage Areas

The Lake Andes watershed is a hummocky, undulating cultivated plain in the Coteau du Missouri physiographic region of South Dakota. The stream network in the watershed is semi-developed and contains numerous small natural wetlands and pockets of potentially non-contributing drainage area during periodically dry climatological cycles. A schematic drawing of the Lake Andes watershed is provided as Figure 2.1. Note the large areas of potentially or generally noncontributing area along the western and eastern boundaries of the upper watershed.

To delineate the combined and individual lake unit contributing drainage areas, the Hydrologic Engineering Center's (HEC) Geospatial Hydrologic Modeling Extension (GeoHMS) for ArcView GIS was used. HEC-GeoHMS is a hydrologic toolkit integrated into GIS with capabilities of documenting watershed characteristics, analyzing spatial data, delineating watershed basins and subbasins, and constructing hydrologic models (primarily HEC-HMS) for engineers and hydrologists. Geo-HMS expedited and simplified the process of watershed delineation in the study watershed. The Geo-HMS process was especially valuable in determining the potentially contributing area in the prairie pothole topography.

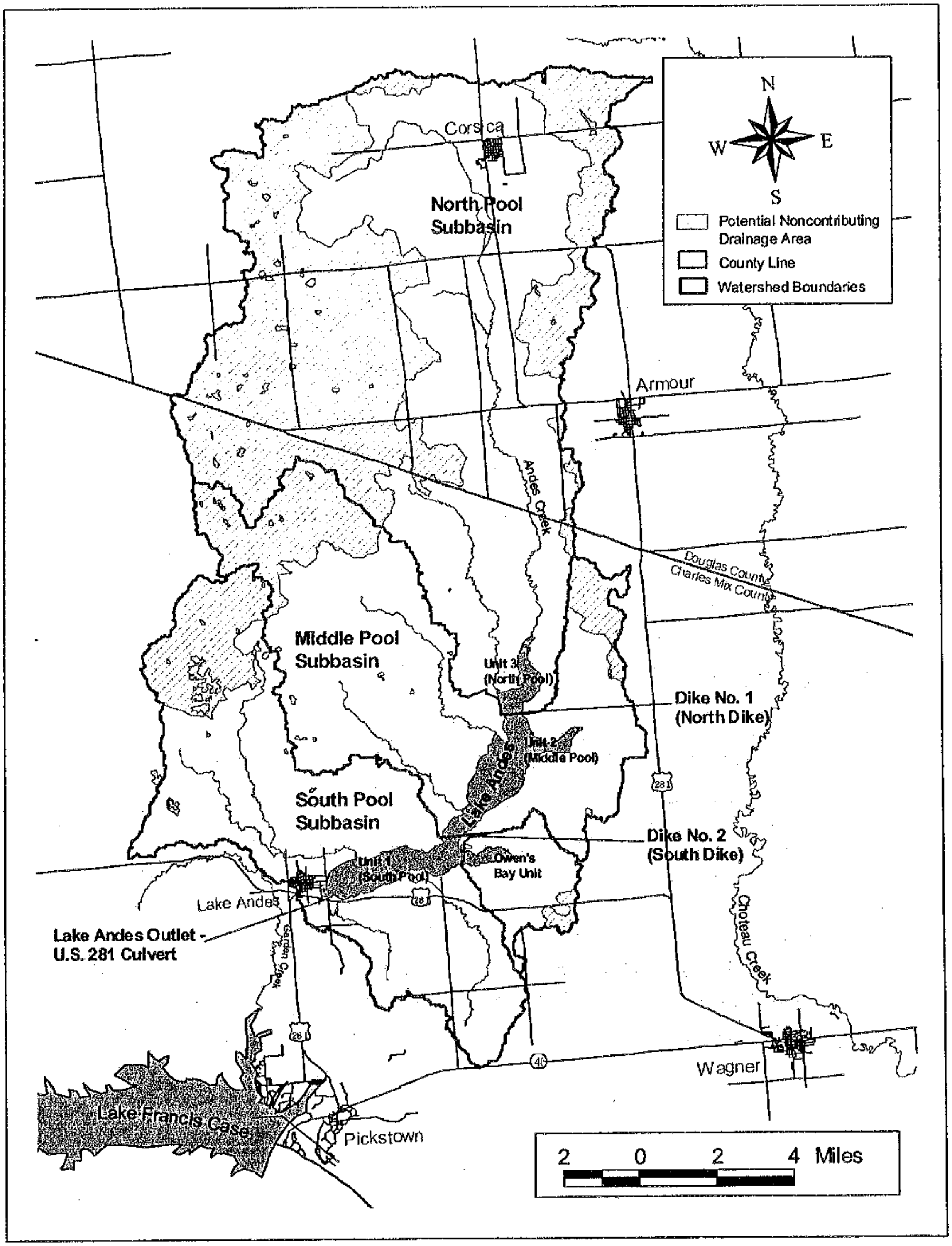

Figure 2.1 Lake Andes watershed and other pertinent study site locations.

GeoHMS delineated the watershed from a composite of digital elevation models (DEMs) constructed from 1:24000 scale USGS DEMs downloaded from a GIS data clearinghouse. The DEMs have a grid cell resolution of 30 meters with elevations referenced to the 1929 NGVD in feet. From the GeoHMS model, watershed basin and subbasin boundaries were adjusted by inspection of 1:24000 scale quadrangle maps of the study area. Additionally, potential noncontributing drainage areas were determined from the quad maps, adjusted based on observations made during the site visit, and digitized into the GIS basin maps to compute final potential and noncontributing drainage areas for the subbasins and the entire watershed. Table 2.2 below provides total potential and generally noncontributing drainage areas for the watershed. During wet cycles, such as in the early 20th Century and again in the 1990s, a greater portion of the potential contributing area yields runoff when sloughs become lakes and isolated lakes spill to Andes Creek.

Table 2.2 Potential Contributing and Noncontributing Subbasin Areas in the Lake Andes Watershed.

Subbasin	Potential Total Area (sq. mi.)	Generally Contributing Area (sq. mi.)	Generally Non-Contributing Area (sq. mi.)
North Unit	123.0	72.9	50.2
Middle Unit	54.6	43.3	11.3
South Unit	50.7	42.8	7.9
Owen's Bay	7.0	6.4	0.6
Total	**235.3**	**165.4**	**69.9**

2.7 Runoff Calculation

WHAM uses a simple soil moisture index (SMI) relationship to determine the fraction of precipitation converted to runoff based on an accounting of soil moisture as an index value. This method is based on the SMI method used in the Streamflow Synthesis and Reservoir Regulation (SSARR) Model. The model requires a minimum of two paired values of the soil moisture index and the runoff factor. A runoff factor is then applied to each inch of precipitation falling on the basin to generate runoff. The index is calculated on a daily basis as

$$SMINDX_{J+1} = SMINDX_J + P - (EVAP(J)*EVFAC(IMON)/100)-(ROFAC*P)$$

in which

$SMINDX_J$	= soil moisture index on the current day
P	= daily precipitation
EVAP(J)	= daily evaporation
EVFAC(IMON)	= monthly pan evaporation coefficient
ROFAC	= runoff factor (fraction) on the current day determined from the index curve.

As the soil moisture index increases, a greater fraction of precipitation is converted to runoff. Excess rainfall (total runoff volume) is determined by multiplying the daily precipitation by the runoff factor for that day's soil moisture index. The runoff factor is directly proportional to precipitation amount, so for precipitation greater or less than one inch, the factor is first multiplied by the rainfall amount to determine the new factor. For example, the runoff factor for a 2.0-inch rainfall would be twice that for a 1.0-inch rainfall. A maximum value for the runoff factor is also specified and was set at 0.50 for Lake Andes watershed. During frozen conditions, daily evaporation is not factored into the soil moisture index adjustment and precipitation is stored as snowpack on the ground surface. When the average daily temperature rises to the snowmelt temperature, snowpack is converted to moisture based on temperature and snowmelt rate. Excess precipitation or snowmelt is then calculated using the soil moisture index-runoff relationship.

To convert rainfall excess to runoff, WHAM distributes rainfall excess according to a daily time step unit hydrograph. Both the initial soil moisture index and the unit hydrograph were taken from the FEMA Hazard Mitigation Study for the Waubay lake Region performed by the U.S. Army Corps of Engineers, Omaha District, and they are shown above in Figure 2.2.

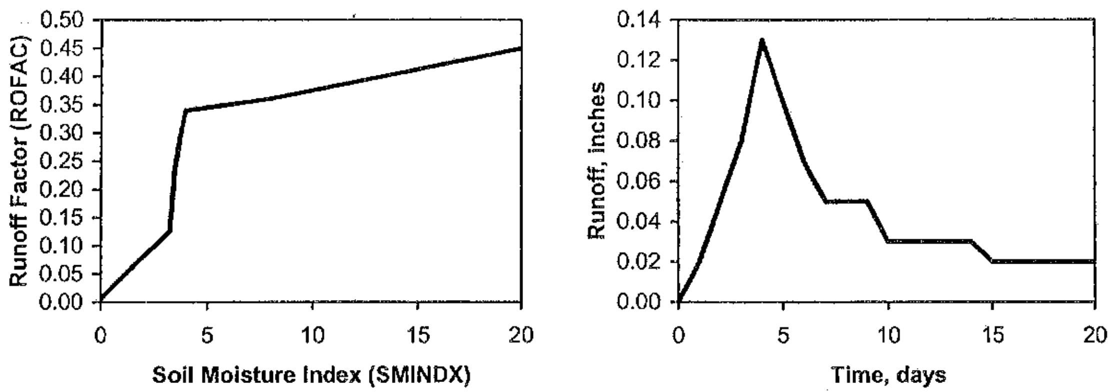

Figure 2.2 Initial Soil Moisture Index Curve and Unit Graph for the Lake Andes Watershed.

2.8 Monthly Hydrologic Parameters

Monthly hydrologic parameters required by the model include seepage rate, evaporation factors, outflow factors, and snowmelt rates. To account for reduced evaporation during colder months, the evaporation factors in November, March, and April were lowered to 0.77, and the values from December through February were lowered to 0.75. Snowmelt rates adopted from the Waubay Hazard Mitigation Study were set at 0.15 inches of water per degree day (F°) from May through October, 0.10 in November & April, 0.07 in December and March and 0.05 in January and February.

2.9 Spillway Discharge Relationships

Outflow rating relationships were determined for hydraulic structures located in the North, Middle and South Lake Units. The U.S. Fish and Wildlife Service provided all information regarding structure types and dimensions. Table 2.3 provides a description of each lake unit's hydraulic structures, dimensions, and crest elevations. Figure 2.3 shows the location of important features including local roads, the dikes and the outlet structures with pertinent details.

Table 2.3 Description of lake unit outflow structures.

Lake Unit	Outlet to Unit	Description	Dimensions	Crest Elev. (ft MSL)
North Unit	Middle	Sharp-crested weir under North Dike low bridge cord	20-ft crest length low cord elev. = 1439.25	1437.25
		7 – corrugated metal pipe arch culverts	58 x 36 inch (span x rise)	1436.40
		Broad crested dike/road crest	2500 ft crest length at variable crest elevation	1441.05
Middle Unit	South	Sharp-crested weir under South Dike low bridge cord	20-ft crest length low cord elev. = 1436.25	1434.25
		Broad crested dike/road crest	3080 ft crest length	1439.25
South Unit	Garden Creek	Normal depth crest under RR bridge	Bottom width = 27.70 Low cord elev. = 1442.95	1437.25
		Sharp-crested weir near inlet of US 281 box culvert	30.5 ft crest length 2 – 10 x 6 ft (span x rise) box culverts, flow elev. = 1433.55, low cord elev. = 1439.55	1437.25
		Concrete subsurface conduit	60-inch diameter concrete culvert	1432.55
Owen's Bay	South	Sharp-crested weir inlet to CMP culvert	5.0 ft weir crest length to 24-inch culvert	1442.42 1440.00

2.9.1 North Pool

Two separate outlets on the North Dike control water discharges from the North Unit to the Middle Unit. In 1942, work was completed on the North Dike and the sharp-crested weir outlet structure having a crest elevation of 1437.25 ft MSL. In 1964 a second outlet structure consisting of seven 58 by 36 inch pipe-arch culverts was completed with an invert elevation 1436.40 ft MSL to alleviate flooding of farmland near the lake.

The free flowing sharp-crested weir flow equation (Eqn. 5-34 of King and Brater) was used to model the 20-foot long, weir crest at elevation 1437.25 ft MSL. Reductions of weir flow under submerged flow conditions were determined using Equation 5-50 of King and Brater. Equations 5-34 and 5-50 are:

$$Q_1 = CLH^{3/2} \tag{5-34}$$

$$\frac{Q}{Q_1} = \left[1 - \left(\frac{H_2}{H_1}\right)^n\right]^{0.385} \tag{5-50}$$

in which Q_1 is free discharge, C is the weir discharge coefficient, L is the crest length, H and H_1 are headwater depths, H_2 is the tailwater depth, n is the exponent from the free discharge equation, and Q is the adjusted submerged discharge.

Since the weir was constructed under the inlet of the North Dike bridge, orifice flow assumes control when the inlet becomes submerged above the low bridge steel elevation at the approximate elevation 1439.25. Two types of orifice flow can occur: 1) un-submerged outlet, and 2) submerged outlet orifice flow. Equations 6.32 and 6.33 of Sturm's Open Channel Hydraulics were used to calculate orifice flow through the bridge opening. The un-submerged and submerged equations of orifice flow are:

$$Q = CA\sqrt{2g(Y - Z/2)} \tag{6.32}$$

$$Q = CA\sqrt{2g\Delta h} \tag{6.33}$$

in which C is the orifice coefficient, A is net area, Y is headwater depth, Z is the hydraulic depth, and Δh is the difference in headwater and tailwater elevations for submerged outlet flow.

A final check for culvert outlet control was performed using the Federal Highway Administration's (FHWA) culvert program HY8. Since the orifice area limited flow through the bridge opening, the outlet tailwater never controlled the North Unit's discharge; and therefore, the controlling flows through the bridge were weir and orifice flow. Discharges were determined for both unsubmerged and submerged outlet conditions using a series of Middle Unit (tailwater) elevations.

To determine the outflow rating of the seven pipe arches, the FHWA program HY8 was used. HY8 uses regression equations developed for FHWA nomographs for various types of culverts. The recommended Manning's roughness coefficient for the corrugated metal culverts is 0.028. Approximately one foot of sediment had settled in the bottom of each culvert, so the roughness coefficient was increased to 0.034 because HY8 does not account for sediment deposition in culverts. A series of Middle Unit (tailwater) elevations was simulated for this outlet to determine the discharge reduction under submerged outlet conditions.

Finally, discharge during road overtopping was determined using the broad-crested weir equation for road overtopping (Eqn. 6.18 of Sturm) for a crest elevation of

1441.05 ft MSL and a crest length of 2500 feet. Submerged and unsubmerged rating curves were prepared for the weir/bridge outlet, pipe-arch culvert outlet, and road-overtopping outlet. For a smoother transition to road overtopping flow, low elevations in the dike were modeled as normal depth sections using Manning's equation. At these sections overtopping flow was calculated while the remainder of the road was regarded as ineffective flow. When the North Dike headwater reached 1442.5 ft MSL, full road-overtopping flow was allowed to commence. The final outflow-rating curves for the structures prior to and after 1966 are provided in Appendix A as Figures A.6 and Figures A.7.

2.9.2 Middle Unit

A weir outlet was constructed through the South Dike in 1942 when the road was built to connect the South Unit to the Middle Unit with the intent that water would flow from the South Unit to the Middle Unit. However, the majority of the water movement since construction has been from the Middle to the South Unit. The structure is a sharp-crested weir at the South Dike bridge opening, identical to the North Dike weir structure, with the crest elevation at 1434.25 ft MSL.

The free flowing sharp-crested weir flow equation (Eqn. 5-34 of King and Brater) was used to predict weir discharge. Reductions of weir flow under submerged flow conditions were determined using Equation 5-50 of King and Brater.

Since the weir was constructed under the inlet of the South Dike Bridge, orifice flow controls when the inlet is submerged above the low bridge cord. Both un-submerged outlet and submerged outlet orifice flow were determined with equations 6.32 and 6.33 of Sturm's Open Channel Hydraulics. Both orifice flow conditions occur at the South Dike Bridge; however since the low bridge cord elevation is low relative to lake levels, submerged orifice flow is dominates.

The site visit revealed sediment deposition to within one foot of the low cord on the north inlet of the South Dike Bridge, suggesting a reduced hydraulic capacity of the flow control structure. Despite the reduced flow area, it was decided that assuming a constant deposition rate, sediment elevations did not affect weir discharge until 1994. In the event that high headwater elevations occur, it was also assumed that discharges through the bridge would scour sediment from the opening allowing the orifice to control releases. The final outflow-rating curve is provided in Appendix A as Figure A.8.

2.9.3 South Unit

The South Unit outlet releases lake water from the combination of lake units to Garden Creek and Lake Francis Case. The original outlet was constructed in 1934 at the congressionally established high water elevation of 1437.25 ft MSL. The present outlet consists of a sharp-crested weir at the inlet of the US 281 box culvert at the same elevation. In addition, an abandoned railroad bridge, its embankment converted to a public trail, has an estimated crest elevation of 1437.25 ft MSL and lies just upstream of the weir and bridge. Downstream of the box culverts lies a 60-inch subsurface conduit which channels all lake releases to Garden Creek. The inlet elevation of the conduit is 1432.55 ft MSL. Backwater created by the conduit can limit Lake Andes discharges.

The controlling outflow rating curves were determined by comparing the weir and box culvert outflow ratings at the highway to the normal depth section at the abandoned railroad bridge and the subsurface conduit outlets. Outflow at the normal depth section under the railroad bridge was calculated using Manning's equation. Sensitivity to Manning's roughness coefficient (n) was tested, and the adopted outflow was based on n = 0.045. The weir/box culvert rating at US 281 is a combination of sharp-crested weir flow (Eqn 5.32 from King and Brater), and culvert flow determined with FHWA's HY8 program. Culvert outlet submergence was dependent on tailwater from the culvert discharge; therefore, it was implicit to the culvert flow calculation resulting in one set of outflow curves. The 16-ft bottom width existing outflow channel condition was considered in determining the tailwater rating curve. The subsurface conduit-rating curve was also determined by HY8. Assuming the conduit was well maintained, the conduit inlet controlled conduit discharge. There is no record of the lake being high enough to overtop the railroad embankment, thus road-overtopping flow was not evaluated for the South Unit.

Comparison of the normal depth section and the weir/box culvert section showed that the normal depth section at the abandoned railroad controlled flow. Comparison of the railroad section curve and the subsurface conduit curve showed that the railroad section curve controls outflow from its crest to elevation 1440.20 ft MSL. The subsurface conduit controls outflow from elevation 1440.2 ft MSL and up (see Figure A.9 in Appendix A). It was uncertain whether the subsurface conduit limited lake outflow to the rated discharges since it was located in the outlet channel several thousand feet downstream of the main lake outlet. Backwater created by the conduit imposes some control on the railroad bridge section discharges. When high lake levels were experienced, the subsurface conduit did not fully control lake discharges because the lake could continue to discharge at rates greater than what is allowed by the conduit. Lake outflow partially bypassed the conduit causing localized flooding near the Bureau of Indian Affairs housing units adjacent to the outflow channel. An exact solution to outflow could be found by performing a steady flow hydraulic analysis on the outlet channel and surrounding floodplain including the subsurface conduit, US 281 culverts and weir, and the railroad bridge section. Due to limitations in the scope of work and budget, we did not perform a hydraulic analysis. Instead, two sets of simulations examined lake levels controlled by the railroad bridge section and a combination of the bridge section and the subsurface conduit. The latter will represent the most limiting outflow condition.

2.9.4 Owen's Bay

The Owen's Bay dike and control structure were constructed in 1936 to allow management of the Owen's Bay arm separately from Lake Andes. An artesian well adjacent to the Owen's Bay arm supplies water to this lake unit. The outlet structure presently conveys water to the Lake Andes South Unit by means of a five-foot long sharp-crested weir inlet to a 24-inch CMP culvert. The weir inlet elevation may be adjusted with stoplogs from its reported crest elevation of 1442.42 ft MSL. The CMP invert was estimated at 1440.00 ft MSL since no design information could be located.

To determine the control structure-rating curve, weir and culvert flow were calculated using the sharp-crested weir equation (Eqn 5.32 from King and Brater) and culvert flow equations in the Federal Highway Department's HY8 program. To simplify the determination, it was assumed that tailwater elevations in the South Unit were low enough to not affect discharges leaving Owen's Bay. This is considered a reasonable assumption since Owen's Bay elevations have averaged above 1440.0 ft MSL. The resulting rating curves are shown in Figure A.10. Weir flow controls to elevation 1443.40 ft MSL then above this elevation yields to inlet controlled culvert flow.

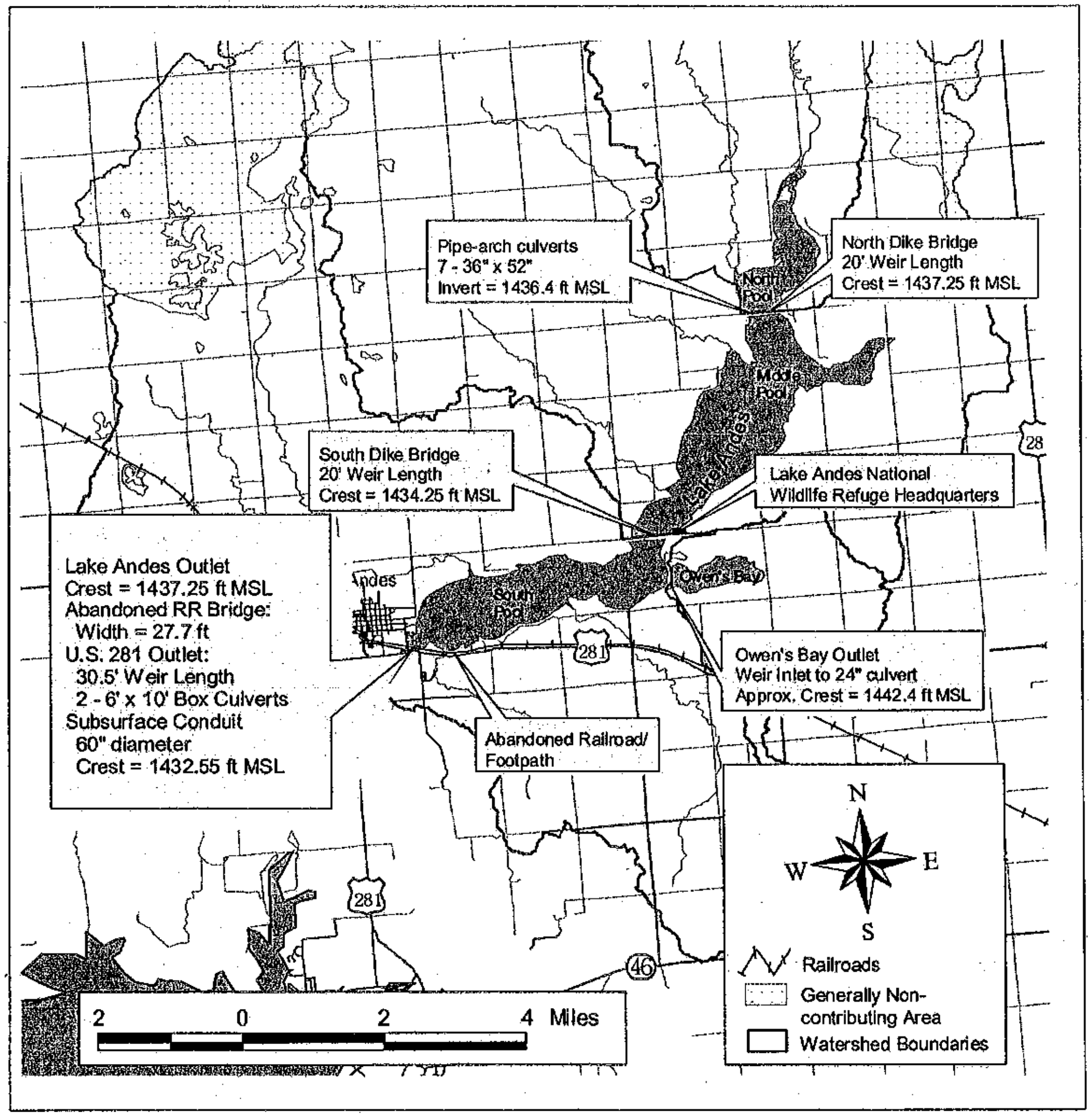

Figure 2.3 Locations and descriptions of Lake Andes lake units, dikes, and outlet structures.

3. Model Calibration

3.1 Subdivision of Model for Calibration

The Lake Andes model was divided into separate time periods for the purpose of calibration. The total modeled period, covers water years 1903 through 2002. The 100-year period was subdivided into four periods, based upon physical changes to Lake Andes and patterns in the lake-level data collected since 1960. The time periods are:

A. 1903 – 1941 Lake Andes without modifications.

B. 1942 – 1966 Three pools subdivided by two roads crossing the lake.

C. 1967 – 1986 Culverts added to north road

D. 1987 – 2002 Three reservoirs begin functioning as 1 pool (Same as Model "a")

Period A. (1903-1941) The lake level data was modeled using the calibrated combined model initially derived for Period D. While there are no known lake level readings for that period, both drought years and high pool years were noted in the "History of Lake Andes". The years 1933, 1934 and 1939 were cited as years when the lake went dry. The model replicated those three years and also produced dry periods in 1935 and 1936 as well.

Period B. (1942-1966) The model was calibrated for the period 1960 – 1966 using data recorded by the Fish and Wildlife Service. The original 3-pool model was derived first for Period C and modified to remove the culverts added in the mid-1960s.

Period C. (1967-1986) The 3-pool model was calibrated for all data available during that period. Reservoir elevation measurements were sparse for the period 1972 – 1982. The model developed is essentially a "current conditions" model and incorporates the hydraulic information obtained on the outlets from each pool.

Period D. (1987-2002) Though Lake Andes continues to be subdivided into 3 major parts by the Dikes; it has functioned as a continuous water surface through most of the recent period. It was calibrated using spot reservoir elevation data collected during the past 16 years.

3.2 Calibration Method

The combined model (Period D.) was the first to be calibrated since it didn't have the complexity of the hydraulic connections between the units found in the 3-pool models. Measured stage information was available for the period 1987 to 2002 and was used as a target for the calibration effort.

Parameters from the Waubay Lakes WHAM Model were used as starting values for the combined model calibration effort. Sensitivity analysis was performed on the WHAM model parameters in an effort to guide the model results towards the measured stage values. Parameters adjusted in the sensitivity analysis included evaporation losses, the Soil Moisture Index, unit hydrograph shape, outflow channel conveyance, outflow invert elevation and whether or not the precipitation came as rain or snow. From the sensitivity analysis, it was determined that the calibration was the most sensitive to the Soil Moisture Index.

Once the combined model was calibrated, a "Period A" record was generated for the years 1903 – 1941. While there were no stage records available for that period, there were historical accounts of when the lake was full and when it was dry. The combined model reproduced the historical droughts and full pool phases without further adjustment.

The next period to be calibrated was "Period C" for the years 1967 – 1986, when the water surface elevations of the 3 units varied separately. Intermittent record was available for that period. The combined model was separated into 3 distinct reservoirs. Owens Bay was not included in this calibration effort as it was found to operate independently of the 3 major units. Spillways were modeled between the 3 units. Rating curves were adjusted to model the elevation-discharge relationship between each unit. During the sensitivity analysis, outflows were restricted to simulate ice buildup on the outlet works. Adjustments were also made to reflect seepage losses and variations in the contributing drainage area between wet and dry cycles. Of the new parameters adjusted, the basin water yield was the most sensitive to outflow ratings and variations in contributing area. Period C (1967-1986) record indicated lower basin water yield than for the years 1942-1966 (Period B). A reduction in contributing drainage area of 35 % had to be applied to the model for Period C. The period 1942 – 1966 used the same contributing drainage areas as were used during the combined model periods. A detailed description of the calibration effort is provided in Appendix B.

3.3 Establishment of Baseline Conditions

The Combined Model, which had been successfully calibrated for the period 1987 – 2002, was used to establish the baseline conditions without further adjustment. Baseline conditions at Lake Andes represent current lake-level and operating trends in the observed data. Since lake levels in the individual units mimicked one another during the period 1987 to 2002, the Combined Model setup and outlet configurations for this period were adopted as the Baseline Model. Outlet configurations in which the railroad bridge section and a combination of the bridge section and subsurface conduit controlled discharges were examined with the Baseline Model. Separate rating curves were prepared for the railroad bridge controlled section and the subsurface conduit inlet. Both curves were used in separate simulations. Comparison of the two showed that lake levels in the subsurface conduit controlled model increase less than 0.10 ft; though, three significant peak elevations were effected by the subsurface conduit and will be discussed further. With the compiled hydrologic databases, 100 years of simulated lake levels were generated with the Baseline Model for statistical analysis. Simulated Lake Andes water levels from the railroad bridge section Baseline Model are shown in Figure 3.1.

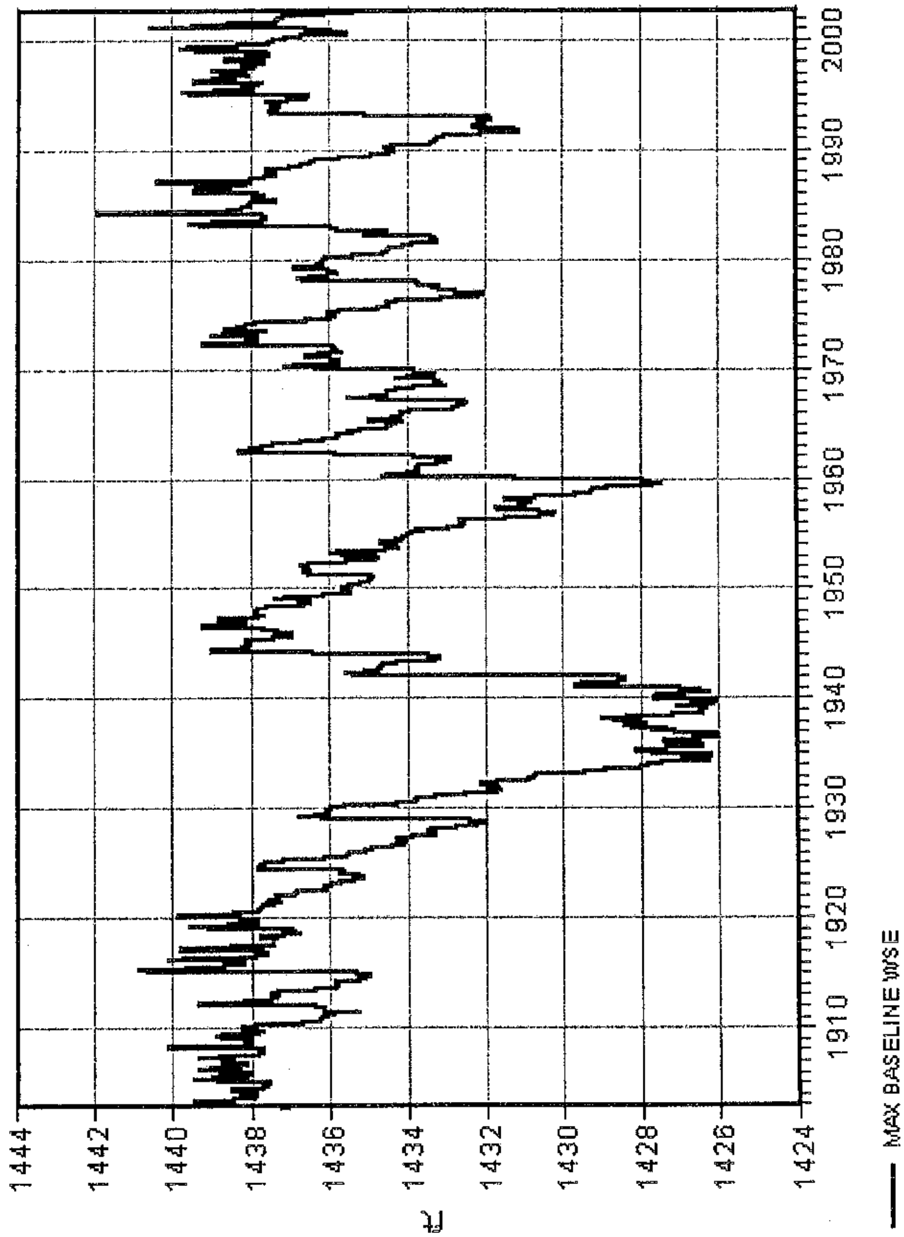

Figure 3.1 Lake Andes Baseline Model simulated water surface elevation from 1902 through 2002.

4. LAKE-LEVEL FREQUENCY ANALYSIS

4.1 Lake-Level Frequency Curve

Lake-level frequency relationships are used to define the annual probability of the lake level reaching or exceeding a certain elevation. Current standards are to express the probability in terms of annual "percent chance of exceedence". For example, a given lake level elevation that has an annual exceedence probability of 0.01 would have a 1.0 percent chance of being equaled or exceeded in any year. The percent chance of exceedence is equal to the exceedence probability multiplied by 100. Once the exceedence probability is estimated, the recurrence interval or return period can be computed as the reciprocal of the exceedence probability. For example, a given lake-level elevation with a 1.0 percent chance of exceedence would have a recurrence interval of 100 years. This means that over a long period of time, the given lake-level elevation would be equaled or exceeded on the average once every 100 years. This elevation would be commonly referred to as the 100-year lake-level elevation. The lake-level frequency curve was derived from the annual maximum lake levels using a graphical frequency analysis with the Weibull plotting position.

From the Baseline Simulation, annual maximum lake levels were obtained from a 100-year period of simulated lake levels. The Baseline Simulation results that incorporated the railroad bridge section outflow control (Adopted Baseline) were used in the main frequency analysis. Weibull plotting positions were calculated from the data and plotted on probability grids. An eye-fit curve was drawn through the annual values to represent the Adopted Baseline lake-level frequency relationship. Weibull positions were also plotted for the subsurface conduit controlled peak lake levels. Figure 4.1 at the end of Section 4 is a probability plot of the eye-fit curve and plotting positions. In general, since the subsurface conduit did not control lake outflows until the headwater elevation reached 1440.2 ft MSL, peak lake-levels were not effected unless they were greater than 1440.2. Four peak lake levels were greater than 1440.2 ft MSL during the 100 years of simulated lake levels. Table 4.1 summarizes lake-level elevations and exceedence frequencies from the Adopted Baseline frequency analysis, and approximate elevations from a cursory frequency analysis of the subsurface conduit control simulation. Elevations from the main frequency analysis were carried forward to the coincident frequency analysis. Subsurface conduit controlled frequency elevations indicate levels the lake could reach if the conduit had total control over lake outflows.

Table 4.1 Lake Andes Baseline Conditions lake-level elevations and exceedence frequencies.

Percent Chance Exceedence	Return Period (years)	Lake Elevation, ft MSL	
		Adopted Baseline	Subsurface Conduit
50.0	2	1436.80	---
10.0	10	1439.70	---
2.0	50	1441.00	1441.75
1.0	100	1441.80	1443.00
0.5	200	1442.20	1443.30
0.2	500	1442.70	1443.80

4.2 Lake-Level Duration

Lake-level duration relationships are used to define the percent of time that a given lake-level elevation is equaled or exceeded. Duration curves represent the cumulative distribution function of all data recorded at the site, which can be based on annual or seasonal periods. Seasonal duration curves can be defined to represent particular months or seasons such as the navigation or non-navigation season. A duration curve is not a probability curve. It should not be interpreted on an annual event basis because it provides only the fraction of time that a given event was exceeded and not the annual probability of an event occurring. It can be used to determine the average number of days per year that a particular magnitude is equaled or exceeded if it is an annual duration curve, or the number of days during a particular month of season if it is a seasonal duration curve. Daily or monthly data can be used to develop a duration curve. A shorter time step in the data used will typically result in a duration curve with steeper slopes at the extremes. Duration curves are developed using class interval analysis. Class interval analysis involves subdividing the data into defined class intervals and computing the relative frequency of each class interval based on the number of data within each class.

A lake-level duration curve was developed from 100 years of simulated Baseline Model daily water surface elevations. The duration curve establishes the length of time that lake elevations will be maintained or exceeded. The lake-level duration curve was generated in HEC-STATS using the class interval analysis. The computed lake-level duration curve is plotted in Figure 4.2 and a summary of the curve is provided in Table 4.2. The lake-level duration curve was used in the coincident lake-level frequency analysis to determine the frequencies that higher lake levels will be experienced. Because of lake level management, high elevations occurred for short periods of time.

Table 4.2 Summary of Lake Andes Baseline Conditions lake-level duration curve.

Percent of time equaled or exceeded	Lake Elevation ft MSL
0.10	1440.90
0.50	1440.00
1.00	1439.60
5.00	1438.80
10.00	1438.30
20.00	1437.80
40.00	1436.70
50.00	1435.80
60.00	1434.90
80.00	1432.80
90.00	1429.60
95.00	1427.20
99.00	1426.70
99.50	1426.60
99.90	1426.40

4.3 Coincident Lake-Level Frequency

Because annual maximum lake levels can be dependent upon the starting lake levels and frequency analysis theory is based on the premise that all data used is independent, an additional analysis was completed to evaluate the effects of starting lake level on the computed frequency curve using coincident frequency analysis. The coincident lake-level frequency relates the probability that events occur coincidently, or the probability that a specified event will occur given another event occurring.

The following steps were used to generate the coincident lake-level frequency curve. Simulations across a range of starting lake-level elevations were performed in which the starting elevation at the beginning of each water year was reset to a constant elevation. Starting lake-levels included elevations 1426.0, 1428.0, 1430.0, 1432.0, 1434.0, 1436.0, 1438.0, 1440.0, 1442.0 ft MSL. A set of lake-level time-series plots is included as Figures C.1 through C.9 in Appendix C. A lake-level frequency curve was generated from each 100-year simulation in the set of simulations, and they are plotted on Figure C.10 of Appendix C. The lake-level duration curve and family of lake-level frequency curves were combined to obtain the coincident lake-level frequency curve.

The resulting relationship is an S-shaped coincident frequency curve similar to the baseline lake-level frequency curve. Both the lake-level and coincident lake-level frequency curves are plotted together in Figure 4.1. The most divergence occurs at frequencies higher than 0.30 and lower than 0.02; otherwise, lake-level frequencies did not vary substantially between 0.02 and 0.30. Table 4.3 summarizes the difference in lake-levels exhibited by each curve at prescribed frequencies. Based on this analysis, the incidence of higher lake levels due to coincident occurrence of high starting lake-levels is very low, because high lake-levels have very low durations. Existing lake outflow structures that limit high lake levels influence the occurrence of infrequent, short-duration lake levels. These structures have the capacity to discharge large volumes of water from the lake when subjected to high headwaters, thus lowering lake-levels, in short periods of time.

Table 4.3 Lake Andes Baseline Conditions Model and coincident frequency analysis lake elevations.

Percent Chance Exceedence	Return Period	------- Lake Elevation, ft MSL -------	
		Adopted Baseline Analysis	Coincident Frequency Analysis
50.0	2	1436.80	1437.30
10.0	10	1439.70	1439.70
2.0	50	1441.00	1441.10
1.0	100	1441.80	1442.00
0.5	200	1442.20	1442.50
0.2	500	1442.70	1443.10

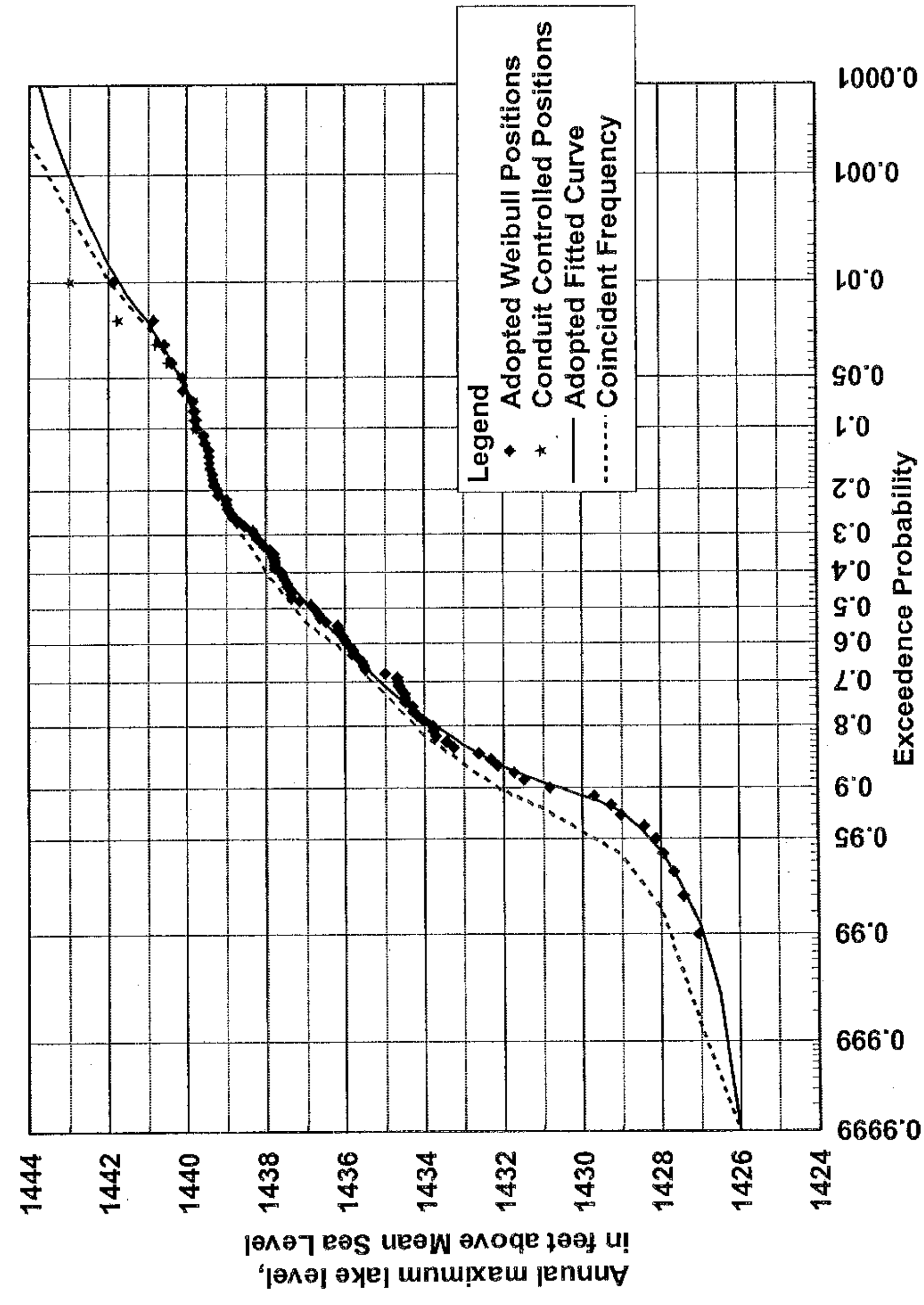

Figure 4.1 Lake Andes Baseline Model lake-level and coincident lake-level frequencies with plotting positions.

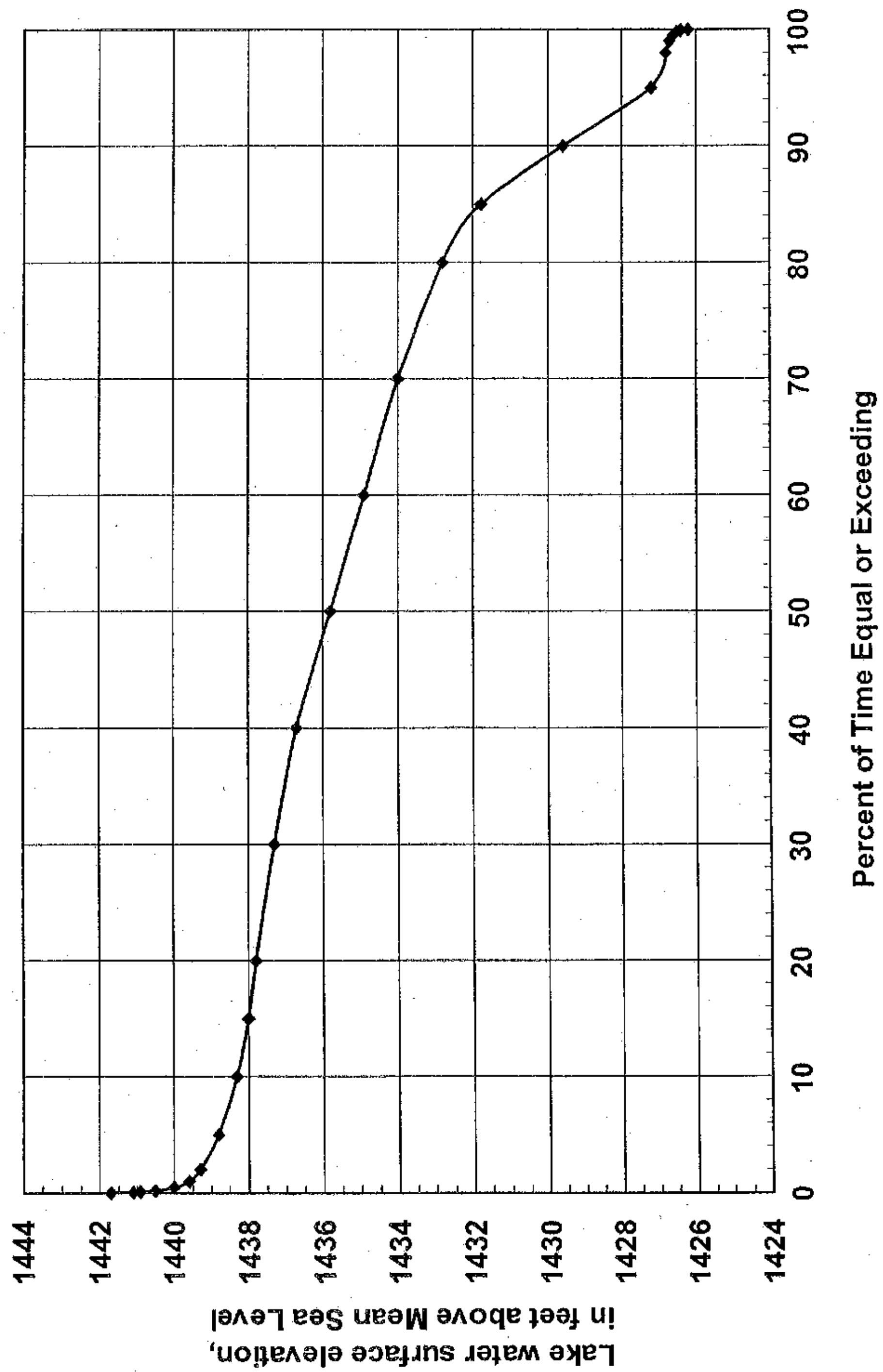

Figure 4.2 Lake Andes Baseline Model lake-level duration relationship.

5. Wind and Wave Analysis

The wind and wave analysis was conducted using standard Army Corps of Engineers methods as outlined in Engineering Technical Letters (ETL) 1110-2-221 (1976) and 1110-2-305 (1984), the Shore Protection Manual (1984) and Engineering Manual (EM) 1110-2-1414 (1989). The Corp's Waterways Experiment Station in Vicksburg, Mississippi developed many of the methodologies presented in those publications.

5.1 Determination of Critical Locations & Lake Levels

The critical locations were determined for each embankment by examining a map of the lake to determine the maximum incident fetch lengths possible on the embankment. The wind-wave parameters were computed on three elevations (1433.25, 1437.25 & 1442.25). Elevation 1433.25 was selected, as there is interest in reconstructing the Lake Andes outlet at Highway 281 with an invert elevation set 4 feet lower. Elevation 1437.25 is the prescribed maximum outlet invert elevation and is considered the normal pool elevation. A weir underneath the Highway 281 Bridge, which has a crest elevation of 1437.25, controls the normal pool elevation. The invert elevation of the channel under the old railroad bridge, located just upstream, is similar. The third elevation selected 1442.25 roughly corresponds to the 100-year lake elevation.

5.2 Critical Fetch Length Determination

The critical fetch lengths were measured at elevation 1437.25 on USGS Quadrangle maps. Radii were constructed from the locations producing the maximum incident fetch. For each fetch, nine radii, drawn 3 degrees apart, were drawn from that critical point to the shore and averaged to come up with the incident fetch length. The direction was assigned according to the central radius of the group.

The fetch lengths for the other elevations were approximated, based upon the amount of lake surface length exposed at that elevation. Lakebed contour elevation maps furnished by the Fish and Wildlife Service were used to define the changes in fetch length applied to elevations 1433.25 and 1442.25. The maps included 2 foot contours below local elevation 102' (elevation 1439.25 msl.) and provided elevation contour 110 as a final maximum elevation. From that information and the pattern of the fetch radii on the quad maps, the fetch lengths were decreased or increased by percentages for the other 2 lake level elevations. Average depths over the principal fetch lengths were also estimated from the lakebed contour maps.

5.3 Wind Probability

Wind frequency tables, developed for the Omaha District Tainter Gate Studies of 1988, were used to compute the design wind. Tabulations of wind probability, by direction, month and for durations of 1, 3 and 6 hours were prepared from National Weather Service hourly anemometer records obtained on magnetic tape for the period of 1948 – 1987, using analysis based upon the log Pearson Type III distribution. The tables provide values for each month and each of 16 compass directions (i.e. N, NNE, NE, ENE etc.) The wind data used at Lake Andes were prepared for the Ft. Randall Dam Tainter Gate Study and used the combined wind records of Sioux City, Iowa and Huron, Pierre and Sioux Falls, South Dakota. All wind velocity readings were normalized to a 33 feet anemometer height using the methods of the Engineering Manual (1110-2-1414). As the lake is frozen during the months of December through February, only the data derived for March – November were used in analyzing wave heights.

Anemometer "waver" was included for this study, in order to select the largest sustained wind from a general direction. The waver factor was incorporated as hourly data from the Weather Service indicates that sustained storm winds often fluctuate a number of degrees. For example, if the predominant storm wind was out of the NNE, higher winds from N or NE may have occurred in the storm and could be substituted for a lower wind from the NNE in computing the wave height.

In the case of Lake Andes, the maximum wind duration considered was the hourly wind, as the fetch-limited waves were developed in less than one hour for all fetch lengths. The design winds for shorter durations were derived from the hourly wind values using relationships published in the cited ETLs and EM. Research (and common experience) indicates that the average wind increases as the duration of the peak wind gust measurement is shortened. Research has also shown that over-water winds are stronger than winds measured at anemometers located on land. As a result, the over land wind values were increased by multiplying them by a factor of 1.2 prior to the wave height computations.

5.4 Significant Wave Heights

Once the design winds were computed, the significant wave height, period and durations were also defined using nomographs or equations obtained from the Corps of Engineers publications previously cited. The significant wave height is used as a standard for analysis and design and is by definition, the average of the top one-third of the waves in a wave train generated by a given wind storm. The wave height is measured from trough to crest. The elevation of a wave above the still water surface would then be one-half the wave height. The duration is the length of time for sustained wind acting on a fetch to develop the maximum waves possible for that storm. The wave period is the time for one wave to pass, measured from crest to crest. The wavelength is the length of wave measured from crest to crest and the 2% wave is the average height of the top 2% of the waves in the wave train, which is roughly 1.4 times the significant wave height.

Depending upon the ratio of the depth of the lake to the wavelength, the friction of the lakebed may act to reduce the wave height. When waves are limited by bed friction, "shallow water" conditions are said to occur. Otherwise they are classified as being "deep water" waves. Conditions are defined as shallow water when the wavelength is more than twice the depth. Both shallow and deep-water wave conditions were encountered on Lake Andes. For Lake Andes, the most severe wave condition occurs on the South Unit where waves of nearly 3.5 feet (setup + runup) could be generated against the west side of the Owens Bay levee by a 100-year storm acting on a full pool. A summary of the wind and wave parameters is provided in Table D.1 in Appendix D.

5.5 Wind Setup & Wave Run-Up

Wind setup is the height above the still pool level to which water is pushed by a sustained wind. The setup is inversely proportional to the depth. It is also a function of wind velocity and fetch length. As a result, shallow ponds with long fetch lengths are more affected by setup, than deep lakes. Due to the shallow depths in Lake Andes, setup is especially severe under a sustained high wind. Winds may effectively push the water off of the windward lakebed while piling up several feet of water on the downwind side of the lake in a 100-year windstorm. The most severe wave setup conditions are found on the South Unit where the water surface may super elevate by as much as 1.5 to 2 feet during a major storm.

Wind run-up is the height above the still pool level to which waves will reach as they run up on a shoreline. Run-up is a function of the wave height & wavelength, the slope of the shoreline and the material of the shoreline. The run-up increases with increasing wave height and can be reduced by making the slope shallower. For the purpose of this study, a riprap covering was assumed for each of the embankments. The approximate slopes of the embankments were estimated during the field investigation. The design run-up is computed as 1.5 times the run-up derived from the significant wave height. Design run-up takes into account the more infrequent and extreme waves within a wave train that can wash out an embankment during longer duration storms. Computed setups and design run-ups are provided in Table D.2 in Appendix D.

5.6 Design Elevations

The design elevation for roadway embankments and the extent of riprap placement around Lake Andes is a function of the water surface elevation, wave setup and wave run-up. The tabulated design elevations are the sum of those three numbers. As indicated in Table D.3 in the Appendix, the elevation to which protection must be provided and the ultimate project cost is very much a function of the lake level. If the amount of time that the lake is at the higher elevations is only a few days every decade, than the risk of an infrequent wind like the 10 or 100-year storm occurring while the lake is high is very small. The top elevation that riprap is placed is also a function of acceptable risk. It may be more cost effective to repair a road or embankment periodically, than it is to extend riprap so high that it is never damaged by wave action.

The most severe conditions are those on the Owens bay embankment, which is exposed to 2.0 to 3.0 miles of fetch to the west. Wave calculations were provided for this location as a reference, as there is no current construction project planned at this location. The trees growing on the lakeside face of the embankment will help break up the force of large waves incident on that shore. The second most severe wave environment is on the Middle Unit. A combination of long fetches, high incident winds and significant setup combine to produce 100-year wave runup impacts more than 5.0 feet above a still water surface of elevation 1437.25 ft MSL on the south side of the North Dike for example. The impact on the north side of the South Dike is slightly less severe. A summary of the final design elevations combining the still pool elevations from the final Baseline Model with wave setup and runup is provided in Table 5.1. Wind-wave runup and setup were analyzed at four locations in Lake Andes. In the table, "Wave Approach" indicates the direction from which the wave is approaching the "Location". This table includes most possible combinations of design wave runup and setup heights compounded on design still-lake elevations. The values represent lake elevations that could occur during the most extreme pool elevations and wind events. Selected combinations of still-lake elevations and wave runup and setup for various lake-level drawdown conditions are discussed in Section 6.3 of this report.

Table 5.1 Summary of existing design elevations combining still pool elevations with wave runup and setup heights.

Location	Wave Approach	------ Still Lake ------		Wave Runup & Setup Elevation for the given Wind Return Period ft MSL		
		Return Period (years)	Elevation (ft MSL)	2-year	10-year	100-year
North Dike/Road	North	2	1437.30	1439.3	1439.9	1440.9
		10	1439.70	1441.8	1442.4	1443.4
		50	1441.10	1443.2	1443.9	1444.9
		100	1442.00	1444.1	1444.8	1445.8
	South	2	1437.30	1439.9	1441.1	1442.4
		10	1439.70	1442.3	1443.4	1444.6
		50	1441.10	1443.6	1444.7	1445.9
		100	1442.00	1444.5	1445.5	1446.7
South Dike/Road	North	2	1437.30	1439.4	1440.3	1441.4
		10	1439.70	1441.8	1442.6	1443.7
		50	1441.10	1443.2	1444.0	1445.0
		100	1442.00	1444.1	1444.8	1445.8
	South	2	1437.30	1438.6	1439.2	1440.0
		10	1439.70	1441.1	1441.7	1442.4
		50	1441.10	1442.6	1443.2	1443.8
		100	1442.00	1443.5	1444.1	1444.7
Owen's Bay Dike	West	2	1437.30	1440.4	1442.1	1443.6
		10	1439.70	1442.9	1444.6	1446.3
		50	1441.10	1444.4	1446.0	1447.8
		100	1442.00	1445.3	1446.9	1448.8
U.S. 281/ Abandoned Railroad	North	2	1437.30	1439.0	1439.6	1440.6
		10	1439.70	1441.4	1442.1	1443.0
		50	1441.10	1442.8	1443.5	1444.4
		100	1442.00	1443.7	1444.4	1445.3

6. Evaluate Future Modified Conditions

The USFWS expressed an interest to manage the lake within a four-foot range for waterfowl production. This would require that lake levels could be lowered as much as four feet below the current invert elevation at the main Lake Andes outlet at U.S. Highway 281 (US 281). Lowering the invert elevation four feet would allow for the maintenance of lower average lake levels and increase the capacity of the main outlet, enabling water volumes to be flushed from the lake system in shorter periods of time.

6.1 Outlet Modifications

Conceptual modifications to the US 281 box culvert outlet included 2-ft and 4-ft lower crest elevations. The existing crest elevation of the outlet is 1437.25 ft MSL at the weir, yet the controlling section of the main outlet is the abandoned railroad bridge (footpath) located immediately upstream of the box culvert. Future modified conditions would require removal of sediment and rock rip rap from the railroad bridge section so that the US 281 culvert and weir could control flow. In addition, it was assumed that the outflow channel downstream of US 281 would be improved to increase flow efficiency and reduce outflow stages. The channel capacity was increased by reducing the roughness coefficient to simulate better maintenance, and the bottom width was increased to 20 feet. Outflow relationships for two possible crest elevations, 1435.25 ft MSL and 1433.25 ft MSL, were determined for the future modified conditions lake level simulations.

A 2-ft lowering of the crest to elevation 1435.25 ft MSL involved the removal of 2.0 feet of stoplogs from the existing weir. The existing US 281 box culverts were not modified. Equations 5.34 and 5.35 for sharp-crested weirs were used to predict outflow under weir controlling conditions. At elevation 1442.10 ft MSL, weir control yielded to outlet control due to high tailwater in the culvert outlet channel. Dual six-foot by ten-foot concrete box culverts were modeled using FHWA HY8 for culverts similar to the computations for the existing conditions. Since the orifice opening also limited culvert discharge, the culvert released slightly lower discharges than unimpeded outlet control for the culvert. The 1435.25 ft MSL crest outlet curve is plotted in Figure A.11 in Appendix A.

A 4-ft lowering of the crest to elevation 1433.25 ft MSL involved the complete removal of all stoplogs from the weir with no modifications to the existing US 281 box culverts. The outflow rating curves for the dual box culverts were determined using HY8, and they are plotted in Figure A.11 in Appendix A along with the rating curve for the 1435.25 ft MSL crest.

The subsurface conduit-rating curve is plotted with the 2-ft and 4-ft lower rating curves. Inspection of the curves reveals that the subsurface conduit gains control of outflows just 1.0 and 0.5 ft above the 2-ft and 4-ft lower crest elevations, respectively. For this reason the outflow limiting effect of the subsurface conduit will be examined in the lake-level simulation and frequency analysis.

6.2 Future Lake-Level Results

From the Lake Andes calibrated baseline model, two future modified conditions models, including the 2-ft and 4-ft lowered crest models with both the regular outlet and subsurface conduit controlling outlet configurations were created. The combined hydrologic record was used to simulate 100-years of lake levels for all future outlet conditions. Lake-level frequency curves were created from the simulated records of lake levels, and they will be used to determine the desired outlet modifications to meet lake-level frequency goals.

6.2.1 Future Lake Levels

Results of the simulations are plotted in Figures 6.1 through 6.2. In each figure, the Future Modified lake level conditions are plotted in a time series alongside the Baseline Model lake levels. Both the regular outlet structure and the subsurface conduit controlling outlet simulations are depicted in Figure 6.1 for the 1435.25-ft crest elevation. The regular outlet structure and the subsurface conduit controlling outlet simulations are depicted in Figure 6.2 for the 1433.25-ft crest elevation. The mean annual peak lake levels for the 100-year simulation period are listed together in Table 6.1. These averages compare to a mean peak baseline lake level of 1436.12 ft MSL.

Table 6.1 Baseline and future conditions statistics of annual peak lake levels from simulated records.

Model Condition	Crest Elevation ft MSL	Mean Peak Lake Elevation ft MSL	Lake Elevation Standard Deviation	Lake Elevation Record Skew
Baseline	1437.25	1436.12	3.40	-0.98
2-ft Lower Future Crest				
Regular Crest	1435.25	1434.06	2.80	-0.78
SS Conduit Limited		1434.27	2.91	-0.67
4-ft Lower Future Crest				
Regular Crest	1433.25	1432.87	2.68	-0.29
SS Conduit Limited		1433.17	2.86	-0.16

In general, lowering the outlet crest elevation lowers mean peak lake levels from 1436.12 ft MSL by 2.06 ft and 3.25 ft for the 2-ft and 4-ft lower outlets, respectively. If the subsurface conduit imposed total control on the Future Modified Conditions outlets, mean peak lake levels would be lowered 1.85 ft and 2.95 ft from 1436.12 ft MSL for the 2-ft and 4-ft lower outlets, respectively. The overall effects of subsurface conduit control on the outlet are higher mean peak lake levels and less efficient peak lake level management.

In observing the lake level plots, it would appear that the subsurface conduit had a limited impact on overall lake levels and almost no impact on the lowest lake levels. Subsurface conduit control resulted in greater increases in lake level elevations when the overall trend in lake elevations was high. Less efficient and lower water discharges through the subsurface conduit had a greater impact on peak lake levels when initial lake levels were high.

6.2.2 Future Lake-Level Frequencies

Annual maximum lake levels of the future modified lake level scenarios were obtained from the 100-years of simulated lake elevations. Weibull plotting positions were calculated from the data, and eye-fit frequency curves were drawn. Figure 6.3 is a probability plot of the eye-fit curve and plotting positions comparing the baseline and the future modified conditions models with the regular (US 281) controlled outlet. The 2-ft drop from current operating conditions causes a substantial decrease in lake-level frequencies because of the increased outflow capacity and the lower crest. A 4-ft crest drop causes an even greater decrease in lake-level frequencies. Table 6.2 below summarizes both baseline and future modified lake levels at prescribed frequencies. A 2-ft outlet drop lowered the 50- and 100-year storms 2.0 and 1.8 feet, while the 4-ft outlet drop lowered the 50- and 100-year storms 2.7 and 2.4 feet, respectively.

Table 6.2 Lake Andes Future-Modified Conditions comparison of lake-level frequency elevations.

Percent Chance Exceedence	Return Period (years)	Predicted Lake Levels ---------- ft above Mean Sea Level ---------- Baseline Conditions	----- Future Conditions ----- 2-ft outlet drop	4-ft outlet drop
50.0	2	1436.80	1434.85	1433.25
10.0	10	1439.70	1436.85	1436.00
2.0	50	1441.00	1439.00	1438.30
1.0	100	1441.80	1440.00	1439.40
0.5	200	1442.20	1440.80	1440.20
0.2	500	1442.70	1441.60	1441.10
			2-ft outlet drop/ conduit controlled	4-ft outlet drop/ conduit controlled
50.0	2		1435.00	1433.55
10.0	10		1437.15	1436.55
2.0	50		1440.20	1439.60
1.0	100		1441.25	1440.95
0.5	200		1441.90	1441.70
0.2	500		1442.50	1442.20

A second set of lake-level frequencies was developed for the future modified outlet condition controlled by the subsurface conduit pipe. This scenario represents the most constrained future outlet conditions releasing lower volumes of water. Both regular and subsurface conduit controlled outlet frequency relationships are plotted in Figure 6.4 for comparison. In addition, Table 6.2 includes the second set of subsurface conduit controlled lake-levels at prescribed frequencies. Conduit control raises lake levels some; but, the differences in lake-levels during shorter return period events (2 and 10-year) are comparatively less than differences for longer return-period events (50 through 500-year).

As stated previously, it is uncertain what degree of control the subsurface conduit has on Lake Andes releases. A general trend observed in this study is that lake levels generally peak at their highest levels (10-year or greater return periods) after either large snowmelt or rainfall runoff events. If the subsurface conduit has total control over Lake Andes outflows, it would have the greatest limits on outflows during high lake levels than during low lake levels, because its rated outflow at high lake levels is much less than the controlling weir and US 281 box culvert. Based on the history of downstream flooding and information from our lake simulations, the exact solution to the lake-level frequency question probably lies somewhere between the presented solutions; however, the safest design elevations are those determined under subsurface conduit control. Recommended still pool lake level frequencies for existing and future management scenarios are provided in Table 6.3.

Table 6.3 Recommended still pool lake level frequencies for existing and future management scenarios.

Percent Chance Exceedence	Return Period	------- Lake Elevation, ft MSL -------		
		Existing Pool-Level Management	2.0-ft Lower Pool-Level Management†	4.0-ft Lower Pool-Level Management†
50.0	2	1437.30	1435.00	1433.55
10.0	10	1439.70	1437.15	1436.55
2.0	50	1441.10	1440.20	1439.60
1.0	100	1442.00	1441.25	1440.95
0.5	200	1442.50	1441.90	1441.70
0.2	500	1443.10	1442.50	1442.20

† Future modified conditions incorporate subsurface conduit flow control.

6.2.3 Outlet Discharge Limitations

Lake-level simulations under the two outlet configurations (limited or not limited by the subsurface conduit) demonstrate that lower peak lake-levels can be achieved while maintaining regular lake levels at a desired elevation such as 1435.25 or 1433.25 ft MSL. The simulations have shown that crest elevation is the primary factor of control. Outflow capacity, however, is very important in managing peak lake-levels because increased outlet capacities will evacuate water from the lake more efficiently.

Table 6.4 compares peak outlet discharges reached at 50 and 100-year lake levels for each outlet configuration and flow-controlling scenario. While the capacity of the US 281 culvert was deemed adequate, outlet channel and subsurface conduit improvements could further lower peak lake levels. For example, in the 2-ft lower management scenario, increasing the channel and subsurface conduit outlet capacity to meet the US 281 rating curve discharge (388 cfs at 1440.0 ft MSL) at the 100-year lake level would lower the peak elevation from 1441.25 ft MSL to 1440.0 ft MSL. These capacities can serve as minimum design requirements for improvement of the US 281 culvert, outlet channel, and the subsurface conduit. It is strongly recommended that the US 281 culvert capacity not be decreased from its existing discharge capacity.

Table 6.4 Lake Andes Outlet peak discharges at associated 50 and 100-year return period lake levels.

Outlet Condition	50-year Lake-Level		100-year Lake-Level	
	Peak Elevation ft MSL	Peak Discharge† Cfs	Peak Elevation ft MSL	Peak Discharge† cfs
Baseline	1441.00	331	1441.80	455
2-ft lower (1435.25 ft MSL)				
regular (US 281)	1439.00	292	1440.00	388
subsurface conduit limited	1440.20	205	1441.25	230
4-ft lower (1433.25 ft MSL)				
regular (US 281)	1438.30	260	1439.40	355
subsurface conduit limited	1439.60	190	1440.95	220

† Peak discharges were reached at the peak lake levels.

6.3 Wind-Wave Events & Future Modified Conditions

Since both the North and South Dike roadway embankments are scheduled for replacement in the future, it was desired to model several different lake operation scenarios in order to determine the economic feasibility of each. By operating the Lake Andes pool at lower elevations, the height of the road embankments and the level to which rock riprap would need to be applied to protect the roadways from wave action could be reduced.

The permanent pool elevation at which Lake Andes is maintained has a lot of impact on how high the roads would have to be built and on the riprap protection that would be required to prevent overtopping or washout. The current condition is a pool that is not allowed to fluctuate significantly from elevation 1437.25. Future scenarios include the possibility of operating the pool 2 feet lower and 4 feet lower under control of the US 281 outlet.

Design elevations were developed using the results of the coincident pool probability distribution and the wind-wave analysis. For the road embankments, the wave setup and run-up are added directly to the pool elevation. Since annual peak stages are not sustained for long periods of time, superimposing the 2-year wind-wave event on the 50-year pool can develop reasonable design elevations for the rebuilt roads. Additionally, the 10 and 100-year wave events can be superimposed upon the conservation pool and each of the proposed future normal pool elevations to provide a range of possible design elevations. Use of the subsurface conduit outlet controlled future lake level frequencies will result in higher design elevations. To obtain these elevations, add the difference between the conduit controlled and US 281 controlled still pool elevations to the elevations presented in Tables 6.5 through 6.7 below. Other pool elevation & windstorm scenarios can be evaluated using data provided in Appendix D.

6.3.1 North Dike Road

The North Dike road separates the North Unit and Middle Unit pools from one another. The road currently has a bridge and 7 culverts that allow water to move between the two units. The elevations listed in Table 6.5 show the minimum elevations to which the road would have to be built and armored. The elevations were evaluated for winds blowing across each of the two units.

Data in the table shows that the embankment would have to be built and protected to elevation 1443.5 in order for the road to remain open during a 2-year windstorm acting on a lake elevation expected once in every 50 years under current operating criteria. The controlling windstorm blows from the south across the Middle Unit.

The table also shows that maintaining the lake at lower elevations would allow the North Dike road to be built and protected to lower elevations for the same frequency storm. If the embankment were consistently operated at elevation 1433.25, then the embankment would only need to be built to 1440.9 to keep the road open during a 2-year windstorm on a 50-year frequency pool.

More extreme winds on more common pool elevations did not require that the road be built that high. Top of road elevations for 10 and 100-year wind storms acting on the current and proposed normal pool elevations for Lake Andes are lower than the rare 50-year pool elevation and a 2-year windstorm.

Table 6.5 Design Wave Setup and Run-up on the North Dike Road Embankment

High Pool, 2-year Wind Storm	Elevation	From North	From South
50-Year Pool (present operation plan)	1441.10	1443.1	1443.5
50-Year Pool (2-foot draw down)	1439.00	1441.0	1441.6
50-Year Pool (4-foot draw down)	1438.30	1440.3	1440.9
Standard Pool Elevations, 10-year Wind Storm	**Elevation**	**From North**	**From South**
Conservation Pool Elevation = 1437.25	1437.25	1439.9	1441.0
2 Foot Draw down Elevation = 1435.25	1435.25	1438.2	1438.9
4 Foot Draw down Elevation = 1433.25	1433.25	1436.5	1436.7
Standard Pool Elevations, 100-year Wind Storm	**Elevation**	**From North**	**From South**
Conservation Pool Elevation = 1437.25	1437.25	1440.9	1442.2
2 Foot Draw down Elevation = 1435.25	1435.25	1439.4	1439.2
4 Foot Draw down Elevation = 1433.25	1433.25	1437.8	1438.2

6.3.2 South Dike Road

The South Dike road separates the Middle Unit and South Unit pools from one another. A bridge opening in the dike allows water to flow between the two units. The elevations listed in Table 6.6 show the minimum elevations to which the South Dike road would have to be built and armored. The elevations were evaluated for winds blowing across each of the two units. The controlling wind storm blows from the north across the Middle Unit and would require the road to be built and protected to an elevation of 1443.3 in order to remain open during a 2-year windstorm acting on a 50-year return period high pool condition. If the permanent pool elevation were to be lowered by 4 feet, a road with a top elevation of 1440.5 feet could provide the same likelihood of road access.

Table 6.6 Design Wave Setup and Run-up on the Railroad Embankment near Highway 281

High Pool, 2-year Wind Storm	Elevation	From North
50-Year Pool (present operation plan)	1441.10	1442.8
50-Year Pool (2-foot draw down)	1439.00	1440.7
50-Year Pool (4-foot draw down)	1438.30	1440.0
Standard Pool Elevations, 10-year Wind Storm	**Elevation**	**From North**
Conservation Pool Elevation = 1437.25	1437.25	1439.6
2 Foot Draw down Elevation = 1435.25	1435.25	1437.2
4 Foot Draw down Elevation = 1433.25	1433.25	1435.3
Standard Pool Elevations, 100-year Wind Storm	**Elevation**	**From North**
Conservation Pool Elevation = 1437.25	1437.25	1440.6
2 Foot Draw down Elevation = 1435.25	1435.25	1438.3
4 Foot Draw down Elevation = 1433.25	1433.25	1436.1

6.3.3 South Unit Outlet

The South Unit Outlet is the beginning of an old overflow channel which ultimately outlets to Garden Creek. Presently the Lake Andes outlet flows under an old railroad bridge followed by the Highway 281 Bridge. A weir under the Highway 281 Bridge is used to maintain the conservation pool at a maximum elevation of 1437.25 feet. As there are plans to rebuild Highway 281, the pool and wave environment at the outlet was analyzed. Under the current operating plan, any structures on the lake side of the old railroad embankment, would have to be protected to elevation 1442.8 feet to avoid damage during a 2-year windstorm acting on a 50-year return period high pool. Operating the pool 4 feet lower would require similar protection only up to elevation 1440.0. The details of the analysis for the old railroad embankment are shown in Table 6.7.

Table 6.7 Design Wave Setup and Run-up on the South Dike Road Embankment

High Pool, 2-year Wind Storm	Elevation	From North	From South
50-Year Pool (present operation plan)	1441.10	1443.3	1442.6
50-Year Pool (2-foot draw down)	1439.00	1441.2	1440.4
50-Year Pool (4-foot draw down)	1438.30	1440.5	1439.6
Standard Pool Elevations, 10-year Wind Storm	**Elevation**	**From North**	**From South**
Conservation Pool Elevation = 1437.25	1437.25	1440.2	1435.4
2 Foot Draw down Elevation = 1435.25	1435.25	1438.4	1437.2
4 Foot Draw down Elevation = 1433.25	1433.25	1436.6	1439.2
Standard Pool Elevations, 100-year Wind Storm	**Elevation**	**From North**	**From South**
Conservation Pool Elevation = 1437.25	1437.25	1441.3	1436.1
2 Foot Draw down Elevation = 1435.25	1435.25	1439.6	1438.0
4 Foot Draw down Elevation = 1433.25	1433.25	1437.8	1439.9

6.4 North Dike Reconstruction and Floods on Andes Creek

One of the design considerations in reconstructing the two roads across Lake Andes is the effect on the roadways of a large peak inflow to the Lake. The most severe condition is a flood on Andes Creek entering the North Unit. The impact of the peak discharge from any of the other watersheds would be minor, as the subbasins are much smaller and the storage much larger for the Middle and South Units. Additionally, the impact of Andes Creek flows routed through the North and Middle Units would have only a minor impact on South Dike or the outflow near US 281.

In order to assess the impact of a flood on Andes Creek, 10 and 50-year flood hydrographs were developed and routed through the existing bridge and outlet structures. An examination of the results from this analysis of existing conditions can be used as a guide to the need for improved hydraulic capacity once the North Dike road is reconstructed. Though short in duration, a flash flood on Andes Creek could close the North Dike Road or wash it out if the road is built too low or the discharge capacity through the embankment is undersized.

6.4.1 Modeling Methodology

The hydrographs used in the analysis were developed using the WHAM model and precipitation records, to simulate 100 years of flow history on the North Unit Watershed. A balanced hydrograph was developed from the daily runoff generated by the continuous simulation. Volume-duration relationships were determined from 100-years of North Unit daily inflow record generated from the WHAM Baseline Model. The volume-duration relationships shown in Figure E.1 of Appendix E were applied to both the 10 and 50-year simulated flood events to develop the hydrograph shape over a several day period. The instantaneous peak discharge of the simulated hydrograph was calibrated to the discharge frequency distribution generated using the USGS Regional Discharge Equations for that part of South Dakota and patterned after WHAM inflow hydrographs. The balanced flood hydrographs are provided in Figure E.2.

A HEC-HMS model was configured to route the peak flow through the North Unit since the WHAM model was limited to daily time steps. Although the total duration of high water can last several days, the Andes Creek watershed peaks much more quickly than 24 hours, requiring the need for the HMS model and its ability to route floods with shorter duration peak flows.

The three rating curves used in the simulations are shown in Figure E.3 in Appendix E. The rating curve for the US 281 controlled condition assumes that the entire lake will raise as a unit and outflow between units will increase only if the total Lake Andes outflow increases.

Two separate starting elevations were used to simulate the flow of a flood on Andes Creek through the North Unit. In the first case, the starting pool elevation was set at 1435.8 ft MSL to represent low lake-level conditions. The routed 50-year flood hydrographs for flow, lake elevation, and storage are shown in Figure E.4 of Appendix E.

The second situation looked at what would happen if storm runoff occurred when the starting pool elevation was at 1441.0 ft MSL, the 2% or 50-year baseline lake level. The first simulation was done using a non-submerged outlet rating corresponding to a surcharged North Unit. The second simulation was done using a partially submerged outlet rating corresponding to a surcharged North Unit. This would be similar to a second storm hitting Andes Creek and the North Unit, before much of the first storm could flow to the Middle Unit. The simulation used 1.0 feet of outlet submergence caused by a Middle Unit pool elevation of 1438.25 feet. The hydrographs for this simulation are shown in Figure E.5.

The next simulation on the high pool elevation was completed using a submerged outlet, reflecting a Middle Unit that was also high, but water moving freely through the Lake Andes system. The third high pool simulation used the US 281 rating curve as the ultimate control for flows moving from the North to Middle Units. The results from the third simulation are shown in Figure E.6.

6.4.2 Results of Analysis

The model was used to compute peak lake levels and discharges through the North Dike during the routing of 10 and 50-year frequency Andes Creek floods through the North Unit. As noted in Table 6.8, peak flows of 860 and 2640 cfs for the 10 and 50-year events were attenuated with a corresponding rapid rise in the North Unit's lake elevations. The 50-year flood overtopped the present North Dike.

It is noteworthy that the draw down of the North Unit Lake by the North Dike hydraulic connections limits the time that the lake can remain at higher levels. Draw down occurs quite rapidly and within two days of the elevation reaching its peak if there is minimal inflow. According to the lake level duration curve, elevation 1441.0 occurs 0.075% of the time or a total time of 0.27 days each year. Therefore the probability of 10-yr and 50-yr floods occurring in coincidence with a 1441.0 ft MSL lake level is extremely low. Note in Figures E.5 and E.6 that lake level draw down from starting lake levels occurs before storm runoff reaches peak lake inflow.

Table 6.8 Lake Andes North Unit peak lake levels and outlet discharges within single flood simulations.

Starting Lake Level ft MSL	Outlet Condition	Peak Lake Levels & Outlet Discharges			
		10% (10-yr) discharge† $Q_p = 860$ cfs		2% (50-yr) discharge† $Q_p = 2640$ cfs	
		Elevation ft MSL	Discharge cfs	Elevation ft MSL	Discharge cfs
1435.8	non-submerged	1438.5	350	1441.4	1310
1441.0	non-submerged	1438.9	470	1441.5	1580
1441.0	partially-submerged	1439.3	500	1441.6	1790
1441.0	US 281 limited	1439.9	390	1444.7	610

† 10% and 2% storm runoffs are imposed on starting lake levels.

The coincident frequency analysis also supports this claim by showing that higher starting lake levels result in nearly insignificant, increases in the lake-level frequency distribution. For example, at the 2% (50-year) flood, the coincident lake level is only 0.2 feet higher than the regular level of 1441.0 feet. At the 1% flood frequency, the elevations are 1441.8 and 1442.0 ft MSL for the regular and coincident lake level frequency, respectively. The greatest lake elevation, 1444.7, in Table 6.8 has a coincident frequency of less than 0.01% or a 10,000-year return interval.

6.4.3 Summary

A 50-year flood superimposed on a starting lake level of 1435.8 ft MSL will overtop the North Dike with the current top of road elevation of 1441.05 feet by a maximum of 0.35 feet. The total overtopping duration will last roughly a day. Higher starting pools result in longer and higher overtopping situations. In the highly unusual situation where the entire Lake Andes system is high (pool elevation of 1441.0 ft MSL and above), the road would be overtopped for a longer time, but the overflow velocity would be lower given the backwater conditions from the outlet at Highway 281.

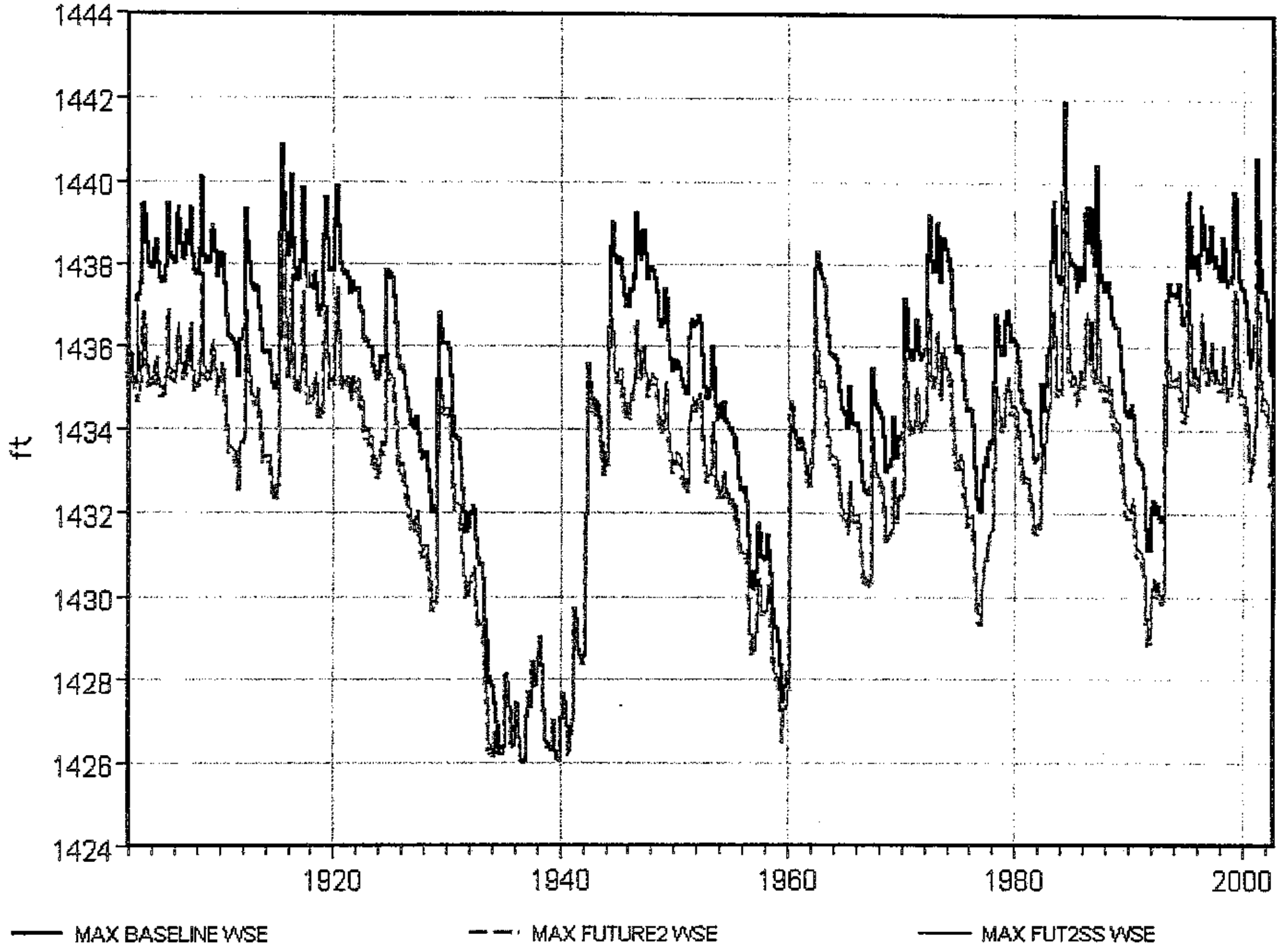

Figure 6.1 Lake Andes Future Modified Model (2-ft lower/1435.25 ft MSL crest) simulated water surface elevations from 1902 through 2002: Comparison of Baseline, US 281 controlled (FUTURE2) and subsurface conduit controlled (FUT2SS) conditions.

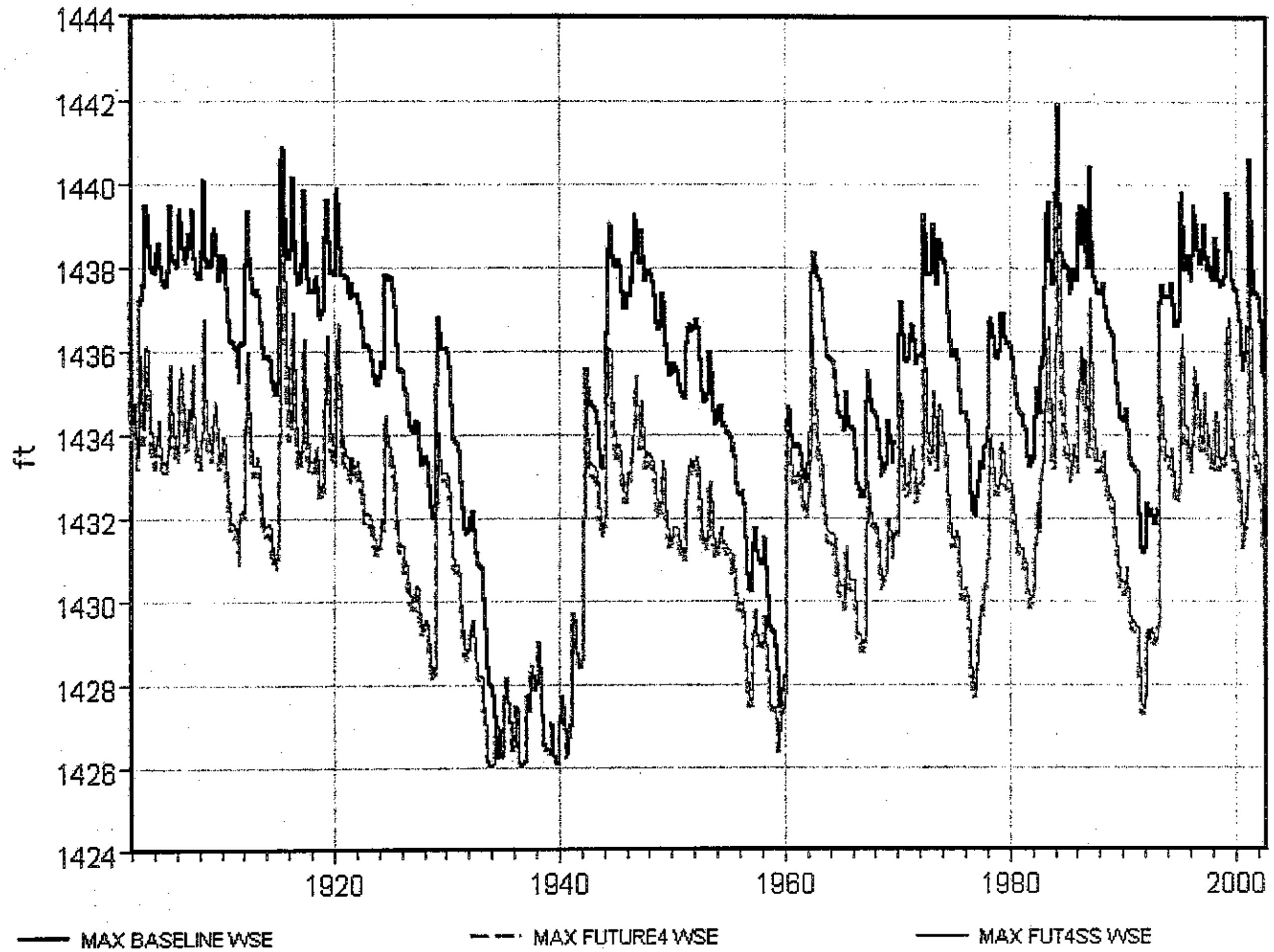

Figure 6.2 Lake Andes Future Modified Model (4-ft lower/1433.25 ft MSL crest) simulated water surface elevations from 1902 through 2002. Comparison of Baseline, US 281 controlled (FUTURE4) and subsurface conduit controlled (FUT4SS) conditions.

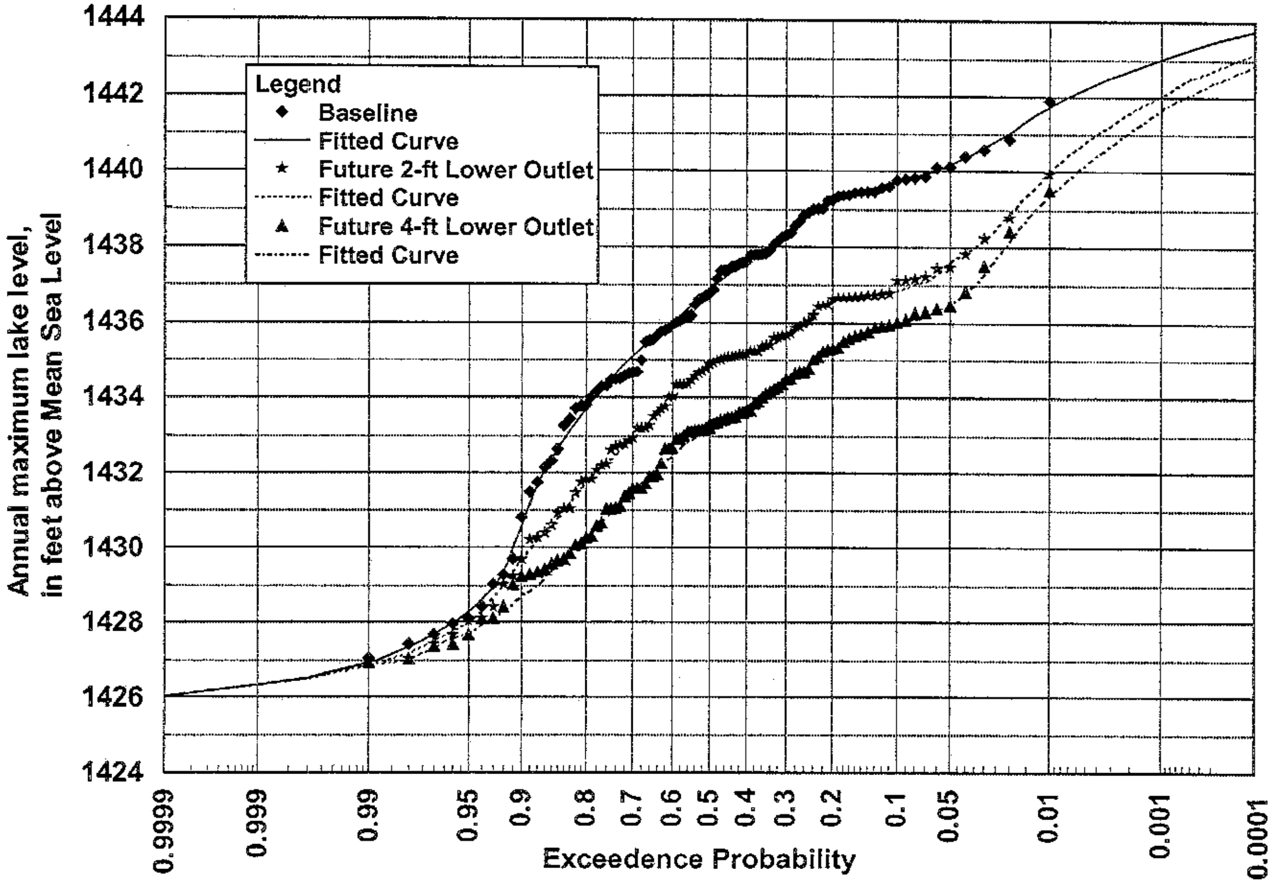

Figure 6.3 Baseline and future modified condition lake-level frequency curves (2-ft and 4-ft lower outlets) and plotting positions.

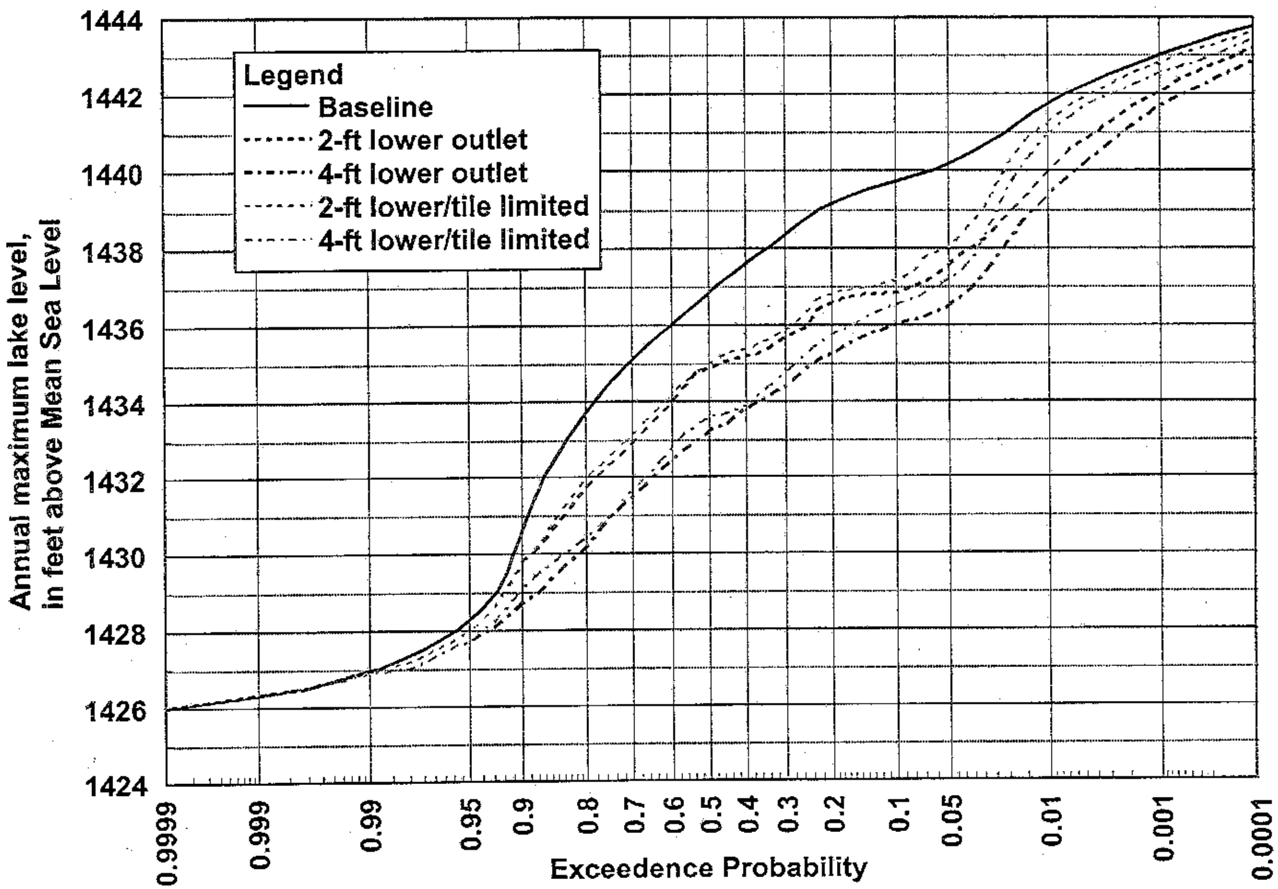

Figure 6.4 Baseline and future modified conditions lake-level frequency curves: Comparison of baseline, future US 281 controlled outlets, and tile (subsurface conduit) limited outlets.

7. PROJECT SUMMARY

Observed lake levels from the period 1987 through 2002 have shown that Lake Andes has recently behaved as a single lake unit with infrequent higher levels at times in the North Unit. Therefore, it is reasonable to use a combined unit approach to model the lake system.

The Adopted Baseline lake level frequency analysis used peak lake levels from the simulation that incorporated the railroad bridge-section controlled outlet. An analysis of simulated lake levels with the subsurface conduit controlling outflows resulted in less than 0.1-ft increases in overall lake levels; however an increase in four high peak lake levels indicates elevations the lake could reach under total subsurface conduit control. Lake-levels of the coincident frequency analysis are slightly higher than lake-levels in the adopted frequency analysis. This difference indicates that starting lake levels may influence peak lake levels; yet, the probability that extreme rainfall events could occur during high lake levels is low.

Maximum coincident lake levels occurring at the 10%(10-year), 2%(50-year), and 1%(100-year) frequencies are 1439.70, 1441.10, and 1442.00 ft MSL, respectively. Existing road elevations reported in NGVD 1929 as-built elevations are 1441.05 ft MSL on the North Dike, and 1439.25 ft MSL on the South Dike, or 1440.25 ft MSL if the raised bridge deck elevation is used on the South Dike. Static pool elevations at the 1% frequency will overtop the dikes while 2% frequency elevations may partially overtop the dikes under current operating conditions.

Future modified conditions of the Lake Andes outlet at the U.S. 281 box culverts could include a 2.0-ft lowering of the weir inlet elevation to 1435.25 ft MSL, or a 4.0-ft lowering of the weir inlet elevation to 1433.25 ft MSL (complete stoplog removal). Operating Lake Andes at lower levels would substantially affect the lake-level frequencies. The downstream subsurface conduit could have much more profound control on future modified conditions lake levels. Maximum lake levels that could occur under subsurface conduit outlet control for the 2.0-ft lower crest elevation at the 10%(10-year), 2%(50-year), and 1%(100-year) frequencies are 1437.15, 1440.20, and 1441.25 ft MSL, respectively. Maximum lake levels that could occur under the 4.0-ft lower crest elevation at the 10%(10-year), 2%(50-year), and 1%(100-year) frequencies are 1436.55, 1439.60, and 1440.95 ft MSL, respectively. Since it is uncertain how much control the subsurface conduit has on Lake Andes outlets, actual future modified peak lake levels could be lower, but it is recommended that the higher future lake-level frequencies in Table 6.3 be considered. Increasing the outlet channel and subsurface conduit capacities could lower lake levels to US 281 culvert controlled levels.

Wave setup, as well as run-up, will cause lake levels to push substantially above static pool elevations during windstorms. Wave action can damage or destroy unprotected earthen embankments. The largest waves for the road embankments are found at the North Dike. A 10-year windstorm on an existing pool at elevation 1437.25

will nearly wash over the road with wave projection to 1441.0 ft MSL. A similar storm projects waves to 1440.2 ft MSL on the South Dike. Waves will wash over the current road embankment, which is at elevation 1439.25 ft MSL. All combinations of lake-level frequencies and wave runup and setup are provided in Table 5.1. Probably combinations incorporating future lake-level management scenarios are provided in Tables 6.5 through 6.7.

A 50-year flood will overtop the North Dike for roughly a day by a maximum of 0.35 feet to elevation 1441.40 ft MSL when the starting pool is at elevation 1435.8 feet MSL. Higher starting pools result in longer and higher overtopping situations. In the highly unusual situation where the entire Lake Andes system is high (pool elevation of 1441.0 ft MSL and above), the road would be overtopped for a longer time, but the overflow velocity would be lower given the backwater conditions from the outlet at US 281.

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APPENDICES

APPENDIX A: LAKE ELEVATION CAPACITY & OUTFLOW RATING CURVES

APPENDIX B. MODEL CALIBRATION

APPENDIX C. COINCIDENT FREQUENCY ANALYSIS

APPENDIX D. WIND WAVE ANALYSIS

APPENDIX E. FLOOD ROUTING ANALYSIS

APPENDIX A. LAKE ELEVATION CAPACITY & OUTFLOW RATING CURVES

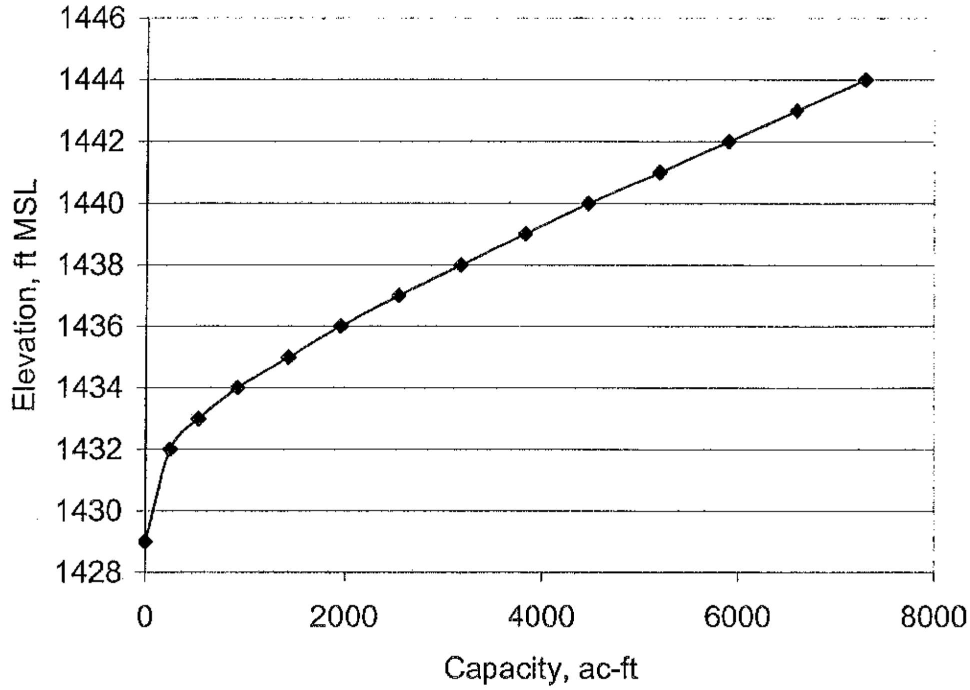

Figure A.1 North Unit elevation capacity curve.

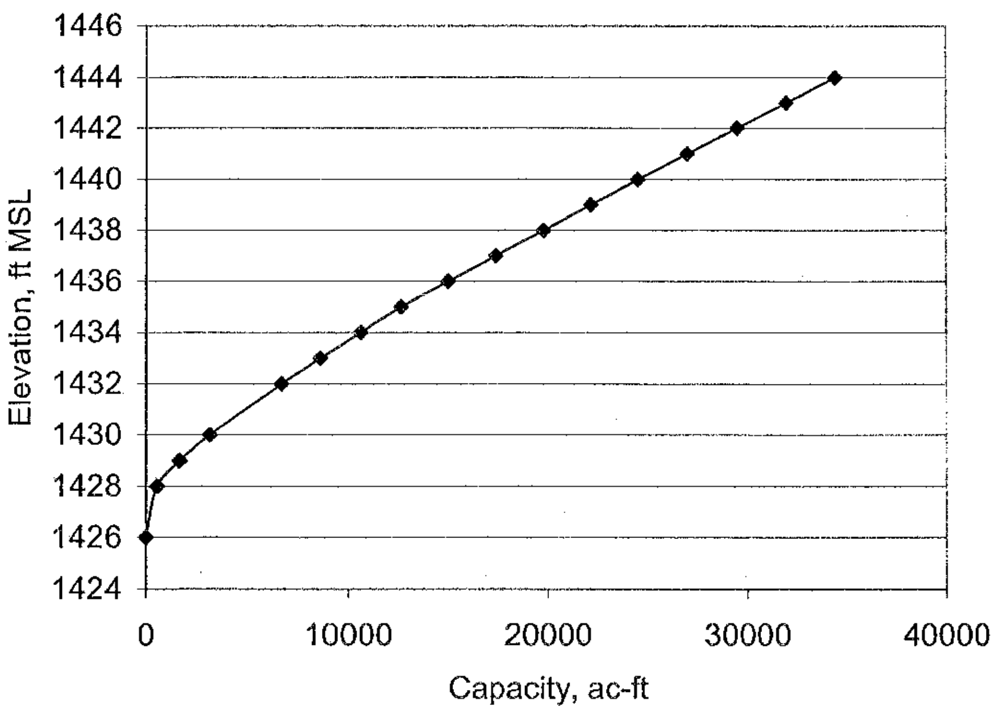

Figure A.2 Middle Unit elevation capacity curve.

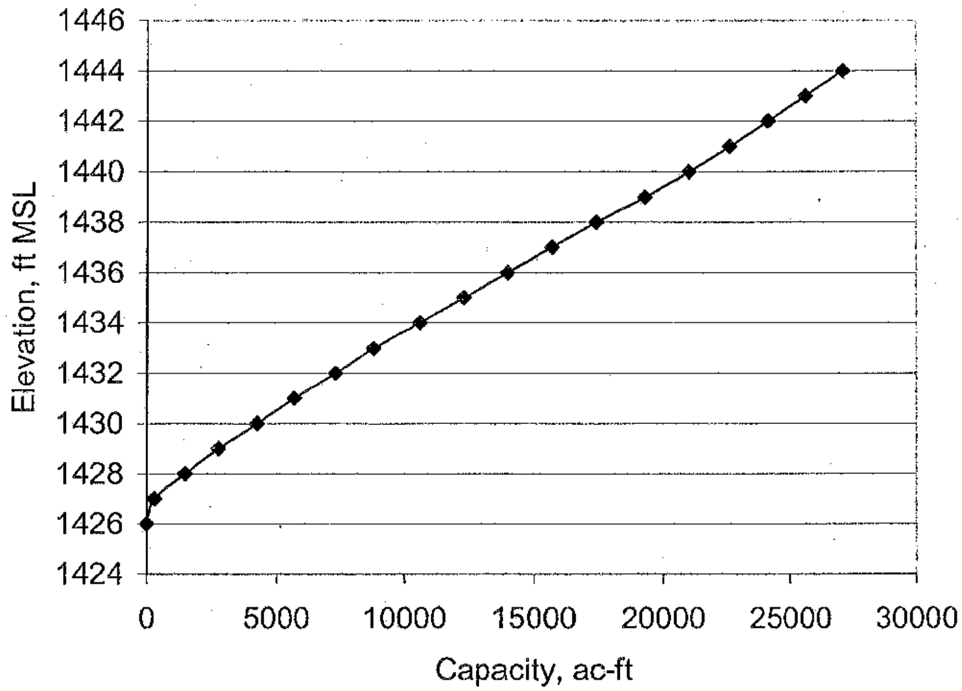

Figure A.3 South Unit elevation capacity curve.

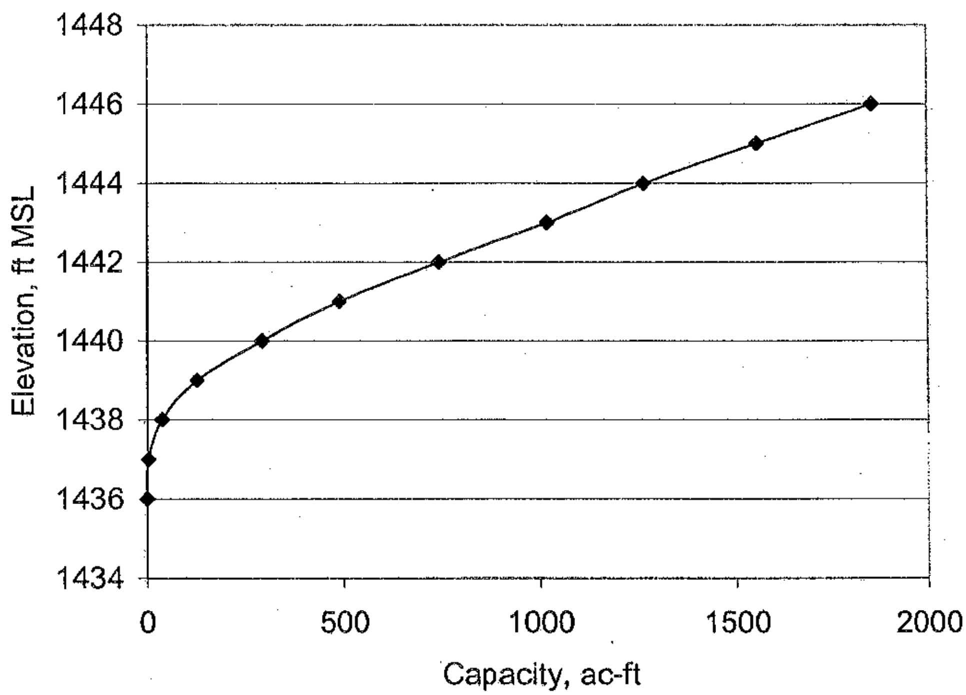

Figure A.4 Owen's Bay elevation capacity curve.

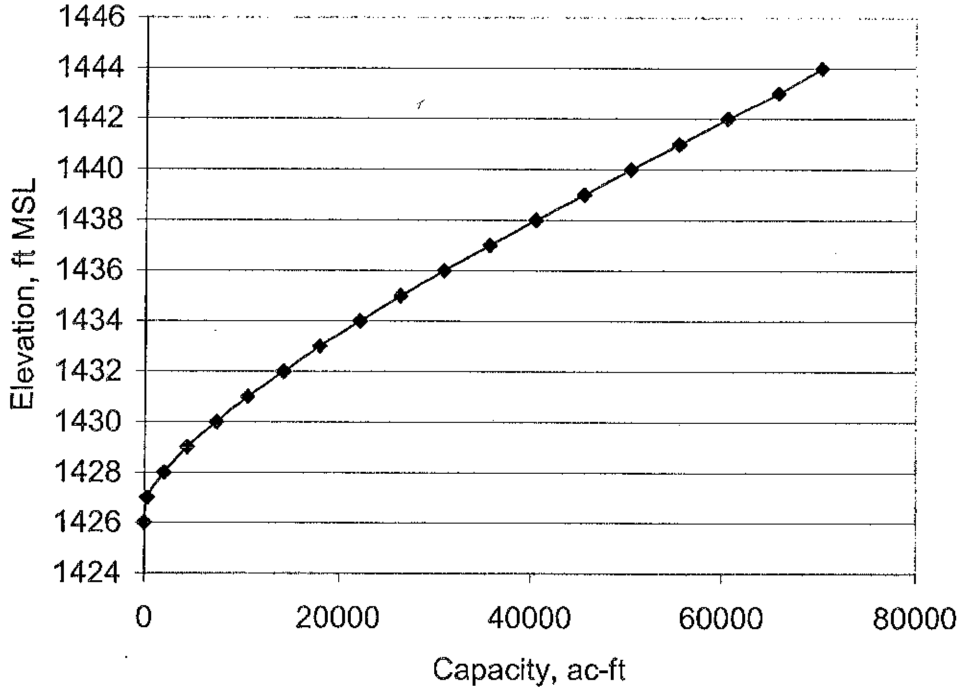

Figure A.5 Lake Andes combined unit elevation capacity curve.

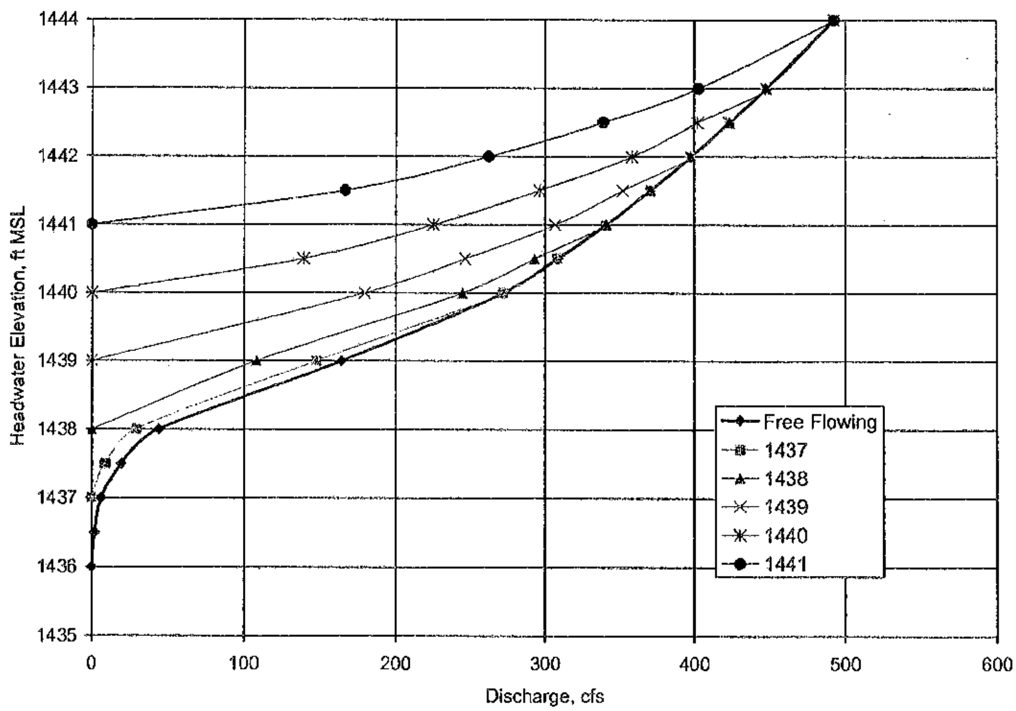

Figure A.6 North Unit weir and bridge free-flowing and outlet submerged outflow curves from 1942 to 1966.

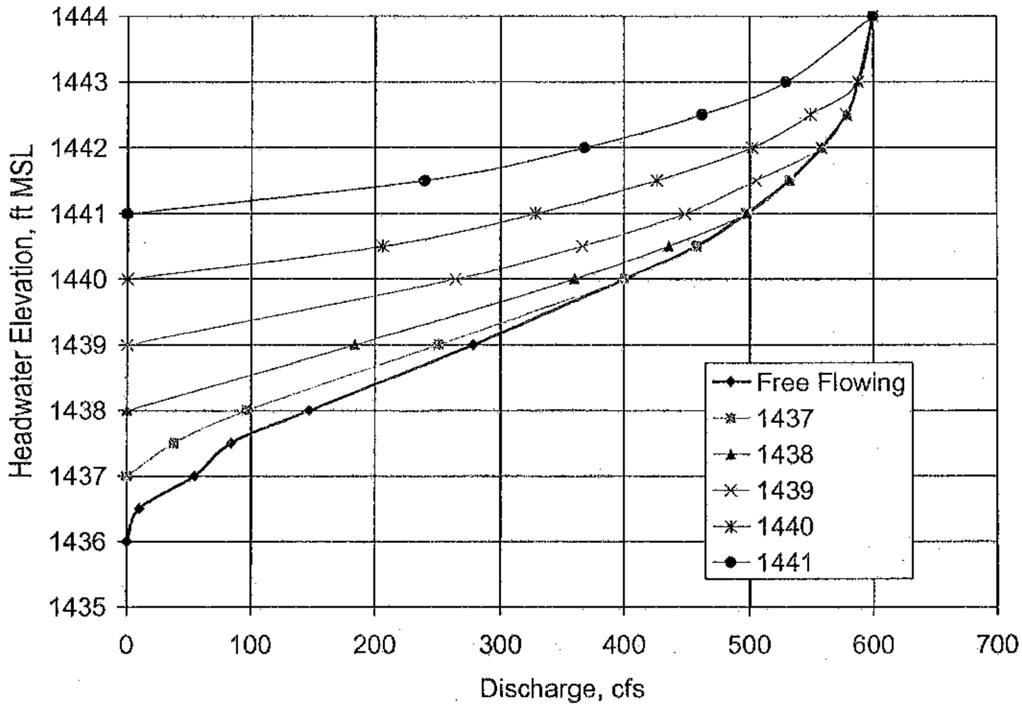

Figure A.7 North Unit weir, bridge, and culvert free-flowing and outlet submerged outflow curves from 1966 to present day.

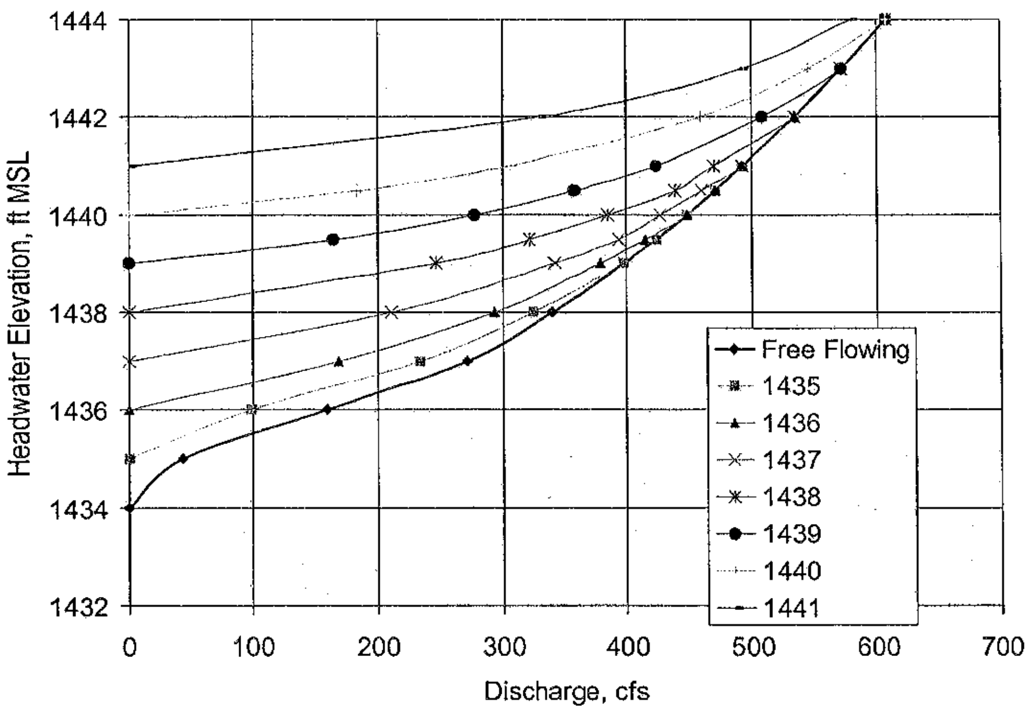

Figure A.8 Middle Unit weir and bridge free flowing and outlet submerged outflow curves.

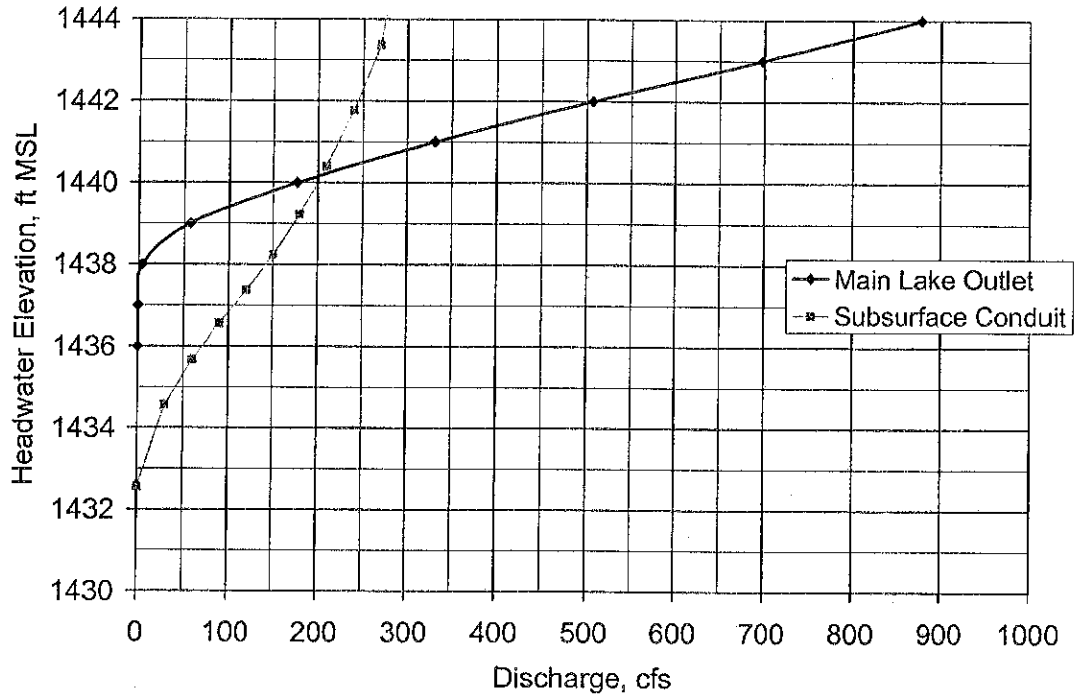

Figure A.9 South Unit and Combined Unit Model calibrated outflow curve.

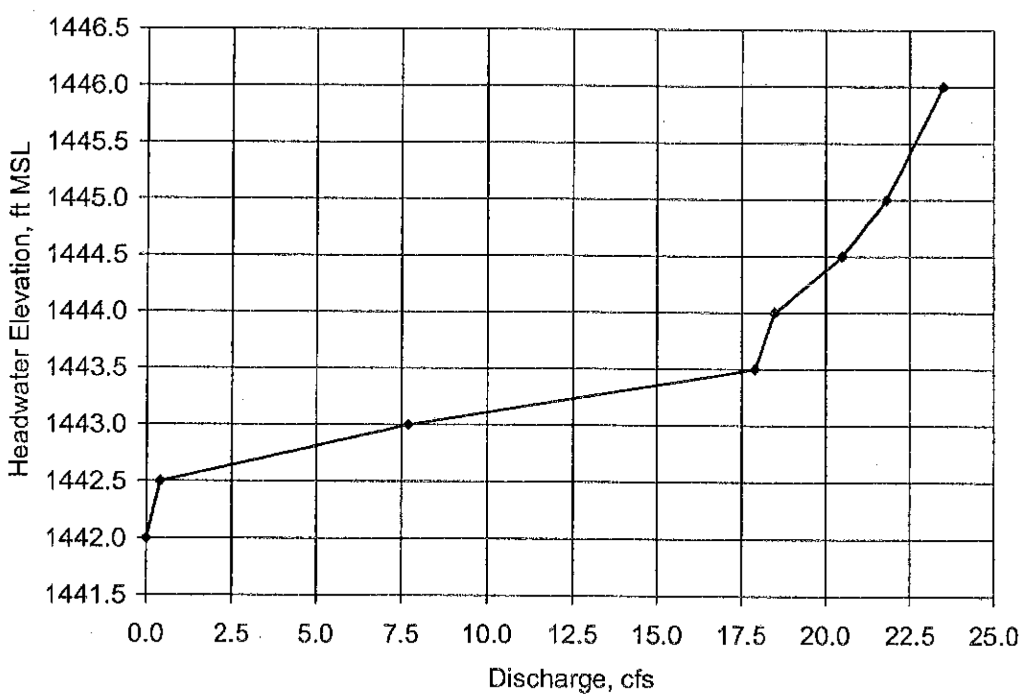

Figure A.10 Owen's Bay outflow curve.

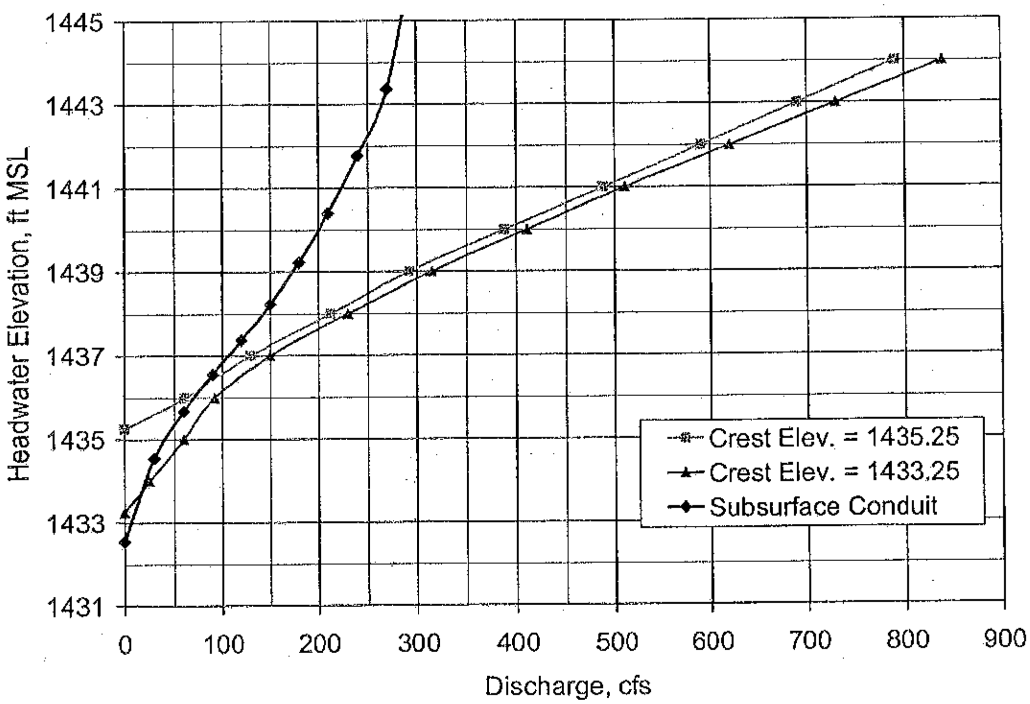

Figure A.11 Combined Unit outflow curves for two proposed inlet representing 2-ft and 4-ft lower crest elevations.

APPENDIX B. MODEL CALIBRATION

B.1 Combined Model

The combined model was the first model calibrated. Since 1987, the measured water surface elevations of the three major units of Lake Andes have tracked closely together. One minor exception was 1993, when the North Unit filled ahead of the other units during recovery from the early 1990s drought. Prior to 1987, the Lake Andes system apparently operated differently, and the stages in the respective units rarely matched each other. Given the apparent difference in operation, the initial calibration effort was made using the combined model for the time period January 1, 1987 through October 31, 2002.

B.1.1 Initial Combined Model

Most of the watershed parameters used in the initial Combined Model were taken from the Waubay Lakes WHAM Model. Among the borrowed parameters were the Soil Moisture Index Curve and the Unit Hydrograph Ordinates. Evaporation pan coefficients were entered, based upon information obtained from National Weather Service Technical Report NWS 33. The stage-storage-outflow rating data were obtained from the Fish & Wildlife Service. The outflow rating for the South Unit at Highway 281 was developed from data obtained during the field investigation in October 2002. The initial model run produced a computed output elevation plot that was many feet above the measured water surface elevations, particularly during the dry years of the early 1990s.

B.1.2 Calibration of the Combined Model

The first parameters to be changed were the monthly evaporation pan coefficient values. They were raised by 10% above the initial values to increase the loss during the dry years. The rationale on increasing the evaporation coefficients was that Lake Andes is much more shallow than reservoirs such as Lake Francis Case for which the coefficients were derived. As a result, evaporation from a shallow pond with a large surface area should be more similar to that of an evaporation pan. That change made a significant improvement in the computed stages. Following that adjustment, changes were made to the slope of the Soil Moisture Index – Runoff Percent relationship (Figure 2.1). Changes in this relationship produced significant results and provided the best tool for calibrating the model. Dozens of model runs were made while fine-tuning the Soil Moisture Index distribution.

Successive adjustments to the Soil Moisture Index brought the computed elevation plot close to the recorded plot, with the exception of the spring 1995 peak. For this event, the computed values remained well below the observed. Additionally, the computed water surface elevations were 0.2 to 0.5 feet too high for much of the 5-year period from 1995 – 2000. Several model parameters were adjusted, in order to analyze the sensitivity of the model to those components. Among the parameters considered in the sensitivity analysis were the unit hydrograph shape (Figure 2.1), the outflow channel conveyance, outflow invert elevation and whether or not the precipitation event came as rain or snow. The model was found to be relatively insensitive to significant changes in those parameters.

Given the good fit to the majority of the record, and the poor fit to the spring of 1995, the accuracy of the model inputs and the recorded stage itself were examined next. The precipitation records for Armour, SD were substituted for the Pickstown, SD records for the March – June 1995 period to determine if a major precipitation event in the Lake Andes Basin had not been recorded by the Pickstown gage. The rainfall record substitution produced little change in the 1995 replication and the precipitation pattern from Armour tended to confirm the Pickstown record. In conjunction with the precipitation analysis, stream flow records from nearby Choteau Creek were examined to verify that region had indeed received a large runoff event from that precipitation. Significant runoff peaks were recorded on Choteau Creek on April 23rd and on May 14 and 30th 1995. To further validate the recorded data, hourly wind records obtained from Ft. Randall Dam were evaluated to determine if wind setup on shallow Lake Andes could have had an effect on the lake elevation readings. Wind data were found for June 2nd, the day of the 1441.3 elevation reading. On that day, the wind averaged 7 mph out of the southeast, with a peak hourly reading of 12 mph out of the southeast. From this information, it was evident that the wind did not materially affect the stage reading.

Following the sensitivity analysis, further adjustments were made to the slope of the Soil Moisture Index – Runoff Percent relationship. Additional slope change points were added to make the slope of the Soil Moisture Index relationship steeper in the range of 3 to 4. This brought the spring 1995 peak up without causing significant deterioration in the model replication of the recorded data over the remainder of the 1987 – 2002 period covered by the combined model. For the drought recovery period of 1993, the computed plot "split the difference" between the 3 pools during the period of divergence. The final results of this calibration effort are shown in Figure B1, for the period 1987 - 2002.

The calibrated combined model was then applied to the period 1903 through 1941. Without further modification, it provided a good qualitative replication of the Lake Andes pool from its glory days as a fishery in the early 20th Century to drying up in the 1930s. Changes have likely been made in the outlet near the town of Lake Andes, both at the rail embankment and at Highway 281. Other than the documentation of the Congressionally authorized settlement that set the elevation of 1437.25 at the outlet structure, there is no chronological record of changes that would have affected the outlet-rating curve. The model results are shown in Figure B2 for this period.

B.2. Three Lake Unit Model

Given the long period when Lake Andes functioned as 3 separate pools (1942 – 1986), the calibrated Combined Model was applied to the 3-reservoir condition, by modeling the hydraulic connections between the reservoirs. This additional work was done to determine if the Combined Model needed further adjustment before being used to compute the baseline conditions.

Two roads were built across Lake Andes in 1941 that became known as the "north" and "south" dikes. The North, Middle and South Units remained hydraulically connected by bridge openings and later a set of culverts between the North and Middle Units. The roads effectively divided Lake Andes into 3 separate pools. Owens Bay, a small eastern arm of the lake, was also separated from the South Unit by a dike. Flows pass from Owens bay to the South Unit via a 24-inch CMP, which can be controlled by stop logs. Since the storage and outflows from Owens bay were small, it was not included in the calibration process. For the most part, the 3 major pools operated independently from the time the roads were built until 1987, when a return to wetter conditions and lake operation changes resulted in a nearly constant water surface between the pools. Since 1987, the pools have tracked together, except for a period in 1993, when excessive precipitation caused a rapid refilling of the pools from very low levels, progressing from North to South.

During the period 1942 – 1986, seven culverts were added to the North Dike to reduce the probability of that road overtopping during a large inflow event. This change resulted in two separate calibration efforts for the 3-reservoir model. The culverts were designed in 1964. While there is no record of the exact date when the work was done, the computed and measured stages for the middle and south pools diverged sharply during 1966 using the model that included only the bridge opening. The fact that the modeled stages of the middle and south pools dropped sharply, but the measured data did not, indicates that more water suddenly became available to those downstream pools that year. As a result, the 3-reservoir model was divided into 2 periods (1942-1966 and 1967-1986) for the purpose of calibration.

The first 3-reservoir model calibrated was for the period 1967-1986, which would be the "current conditions" model, if the reservoirs hadn't begun to act as a single water surface after 1987. This model includes the current hydraulic connections between the pools. The calibration began using the model parameters of the combined model and the rating curves developed for the openings in the north dike and south dike. The rating curves in the model were a composite of the flow through the 7 culverts and bridge at the north dike and the bridge at the south dike. The rating curve for the outflow from the South Unit through the railroad and Highway 281 bridges was also included in the model.

The initial efforts to calibrate the 3-reservoir model resulted in instability between the 3 pools due to the Modified Puls routing method. The instability was traced to the fact that large quantities of water can theoretically move between the pools when they are at high stage in one day. According to the model, a sizable portion of the total North Unit storage could relocate to the Middle Unit in a single 24-hour period. The WHAM model does not permit time steps shorter than 1 day; so using a shorter time step could not eliminate the instability. This limitation made it hard to achieve a stable water surface in all of the pools during flood events. In practice, however, the large daily water transfers do not occur due to the of backwater limitations placed upon the north and south dike flows by the system outflow at Highway 281. The larger hydraulic capacity through the north dike is similarly limited by the constricted flow through the south dike. To resolve this problem, a backwater-rating curve was developed for the North Dike outlets that permits unrestricted flow at lower elevations, but gradually conforms to the outflow rating of the Middle Unit through the South Dike at higher lake elevations. The model became stable and produced results which were higher than the stages recorded for the 3 pools from 1967 through 1986, once the backwater rating curve was implemented for the North Dike. The stages on the North Unit were slightly high while the computed elevations for the Middle and South Units exceeded the observed values.

Since the predicted water surface elevations were too high, changes were made in the model to reduce the amount of water in the 3 units. The first change made as part of the continuing calibration process was to restrict flows during winter months. Ice buildup on the weirs beneath the bridges and in the corrugated metal pipe culverts reduces the flows during the months of November through March. This change brought the computed stages down slightly, for the lower units but didn't significantly close the gap between the computed and measured values.

Following that, the effect of seepage losses from the system was evaluated. When a seepage loss in inches per day is applied to a daily record, the loss is removed from the system simulating a percolation to a ground water table from a perched wetland. Various seepage losses were applied to each of the 3 units for the period 1967-1986. A constant seepage rate was applied to all 12 months. The best fit occurred when the following pond seepage rates were used, but the replication of the measured data was only fair at best.

North Unit = 0.050 inches / day
Middle Unit = 0.085 inches / day
South Unit= 0.050 inches / day

The seepage rates were then applied to the period 1942-1966. The computed lake elevations did not provide a good replication of known information and stage data for the period. It produced dry periods for the Middle and South Units in the 1950s that were much longer than the 2 dry spells noted in 1958 and 1959. Given the rather poor results, reservoir losses to seepage did not appear to be a factor in the hydrologic balance of Lake Andes.

Drainage area was the next factor evaluated in an attempt to define and explain the pattern of recorded lake elevations in the 3 units during the period 1942 – 1986, when the pools behaved independently. Historically, the drainage area of the Lake Andes watershed has varied, with the contributing area fluctuating with climatic cycles as is typical in the prairie pothole topography of the northern plains. Throughout the eastern Dakotas, the drought of the 1930s left a strong imprint on the hydrology of the glacial lake region in the 20th Century. This phenomenon has been studied at other locations, most recently at the Waubay Lakes system in northeast South Dakota. The pattern features a normal to wet period in the first 3 decades of the century followed by the drought, and a slow recovery lasting into the 1980s, followed by a wet period to round out the 20th Century.

Lake Andes evidences a similar pattern. From 1894 to 1933, there is no record of the lake going dry. It flourished as a bass fishery in the early 1920s and was over 18 feet deep and 14 miles in length during that period. The high water levels flooded surrounding farmland resulting in an outlet tube ultimately being constructed in 1934 when the lake was dry. The lake fluctuated between wet and dry cycles after that, without a return to a lengthy period of sustained high water until after 1993.

A good fit of the measured lake elevations and the pattern of dry years was obtained for the period 1942 - 1966 without using a reduction in drainage area. The results of this calibration effort are shown in Figures B3, B5 & B7.

For the period 1967 until 1986 when the pools began to operate in unison, a better fit to recorded data was obtained by reducing the contributing drainage areas of the North, Middle and South Unit watersheds. Severe drainage area reductions produced the best fit. The following contributing drainage area reductions were made:

North Unit area reduction	= 35%
Middle Unit area reduction	= 60%
South Unit area reduction	= 35%

The reduction applied to the Middle Unit in particular looks particularly excessive. It is possible that in addition to a reduction in the contributing drainage area that the operation of the stop logs at the two embankments also contributed to the lower than predicted water surface elevations. Absent a record of the timing and magnitude of the stop log changes, the three-pool model for the period 1967 - 1986 could not be further calibrated. The calibration results are shown in Figures B4, B6 & B8 for the 3 major units.

The watershed and reservoir parameters obtained from the calibration process on the 3 major units were applied to the small Owens Bay unit. The computed water surface elevations provided a fairly good match of the measured water surface elevations for the period 1960 through 2002. The greatest differences between computed and measured water surface elevations were grouped in a few years. The unexplained variability in those years may be due to the operation of Owens Bay using the stop logs in outlet structure to store or evacuate water from the impoundment. The calibration results for Owens Bay are shown in Figures B9 and B10.

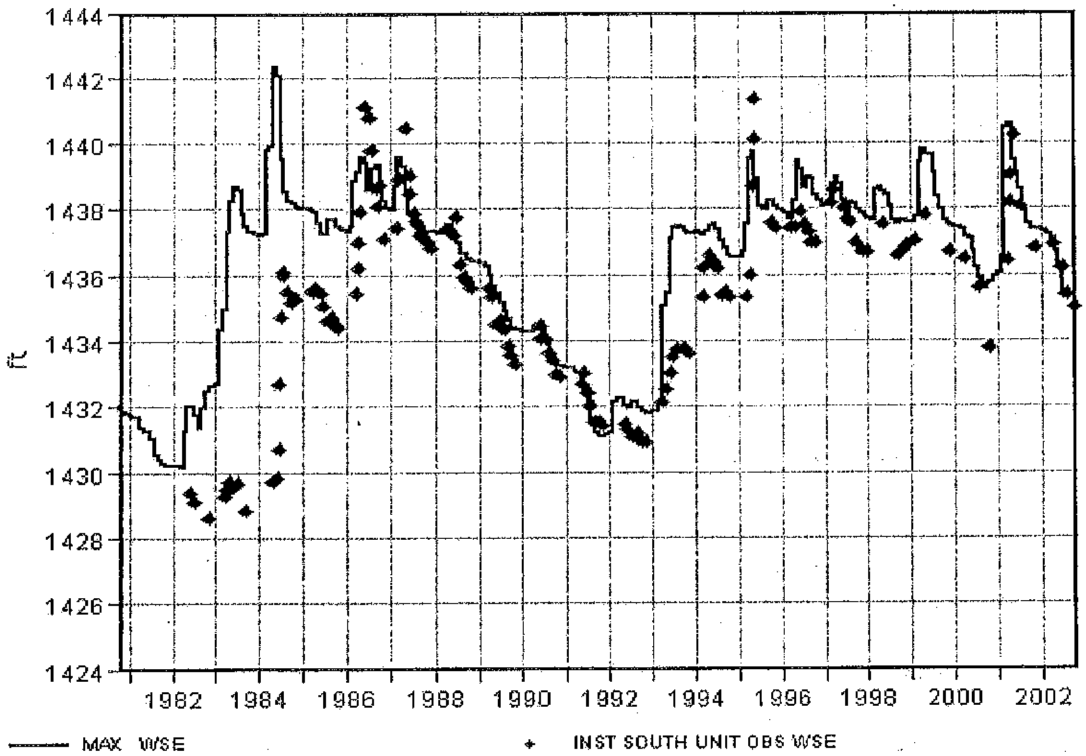

Figure B.1 Lake Andes Combined Unit simulated maximum water levels (solid line) and observed water surface elevations (symbols), from 1981 through 2002.

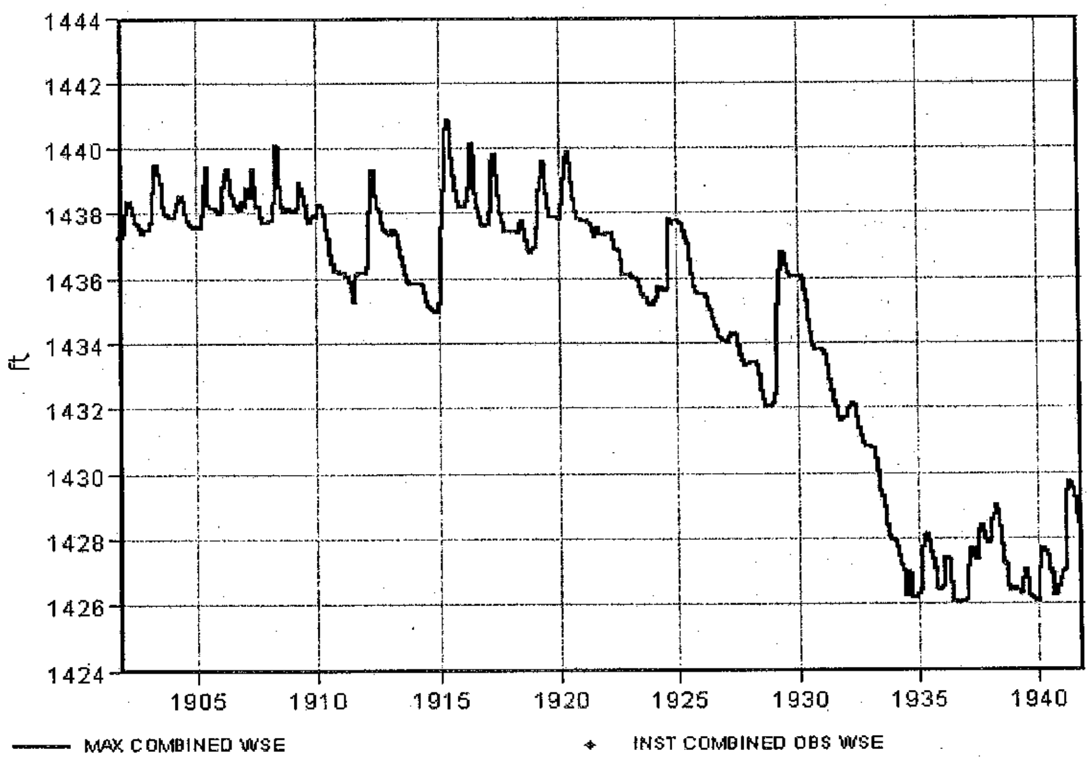

Figure B.2 Lake Andes Combined Unit simulated maximum water levels from 1902 through 1941.

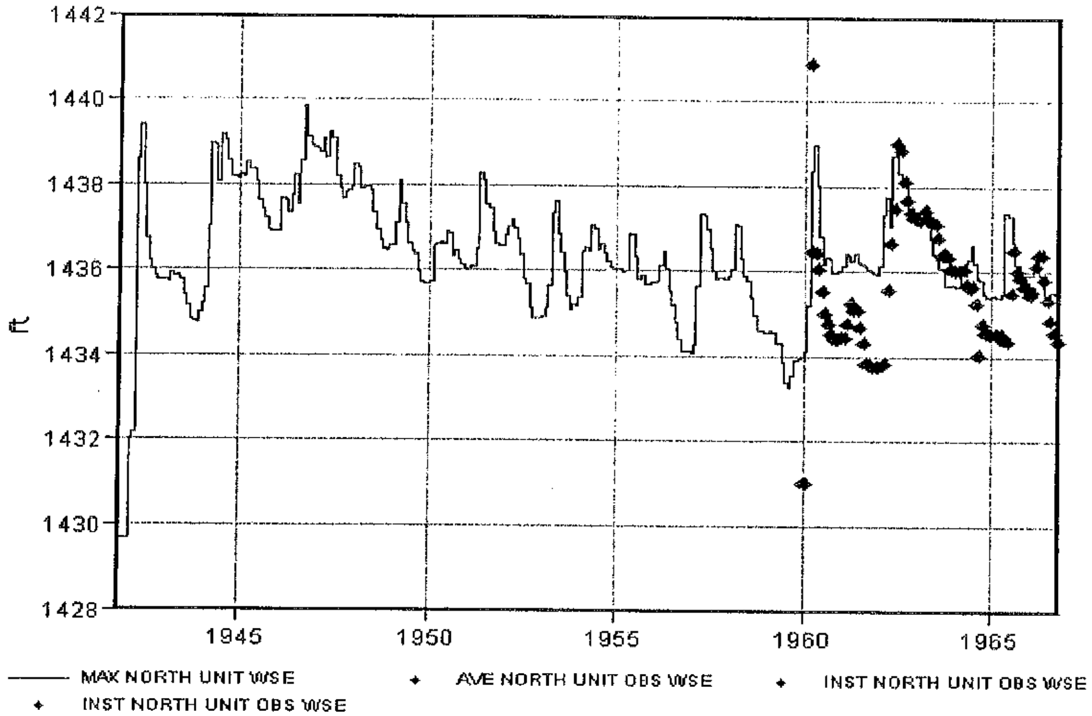

Figure B.3 Lake Andes North Unit simulated maximum water levels (solid line) and observed water surface elevations (symbols), from 1960 through 1966.

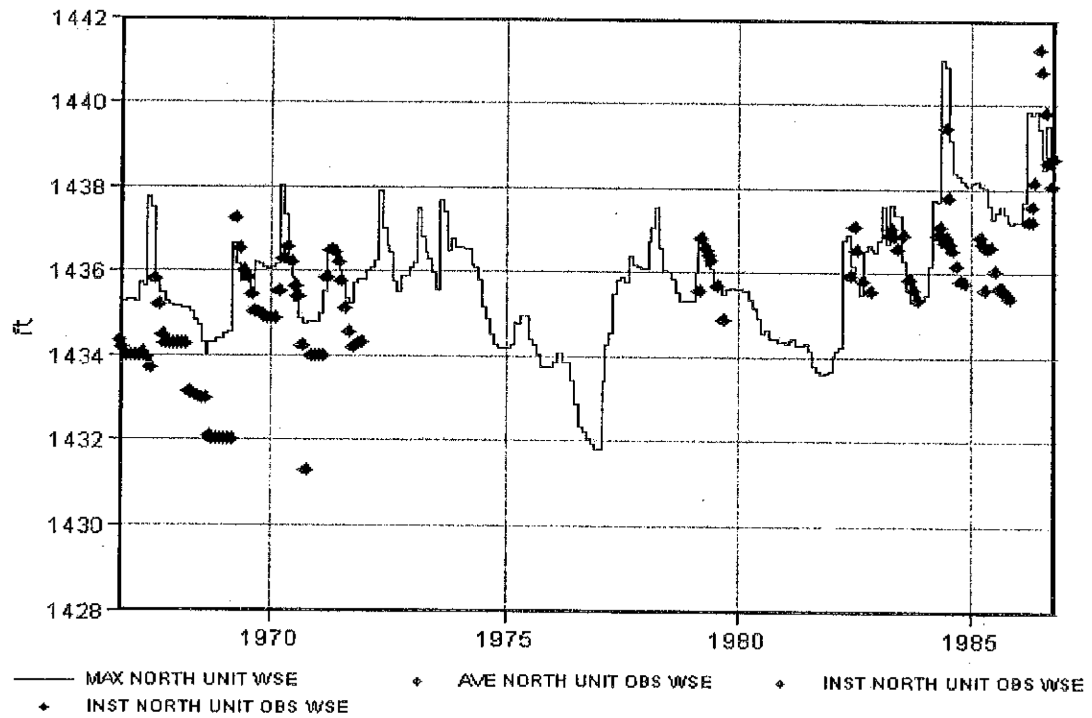

Figure B.4 Lake Andes North Unit simulated maximum water levels and observed water surface elevations from 1967 through 1986.

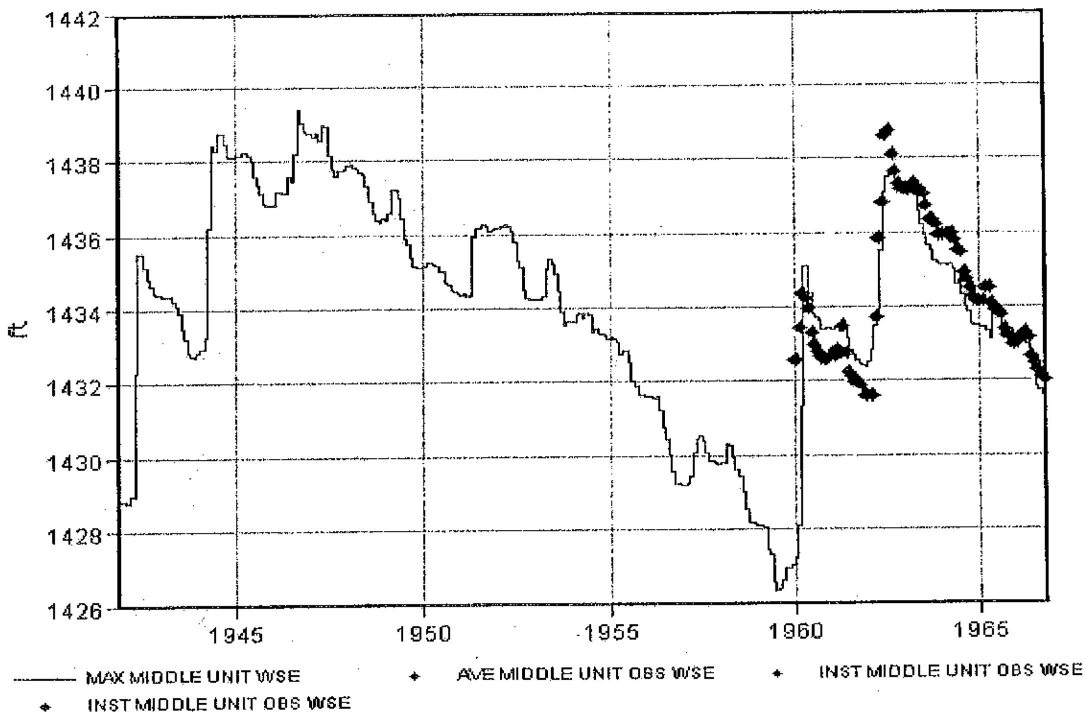

Figure B.5 Lake Andes Middle Unit simulated maximum water levels and observed water surface elevations, from 1942 through 1966.

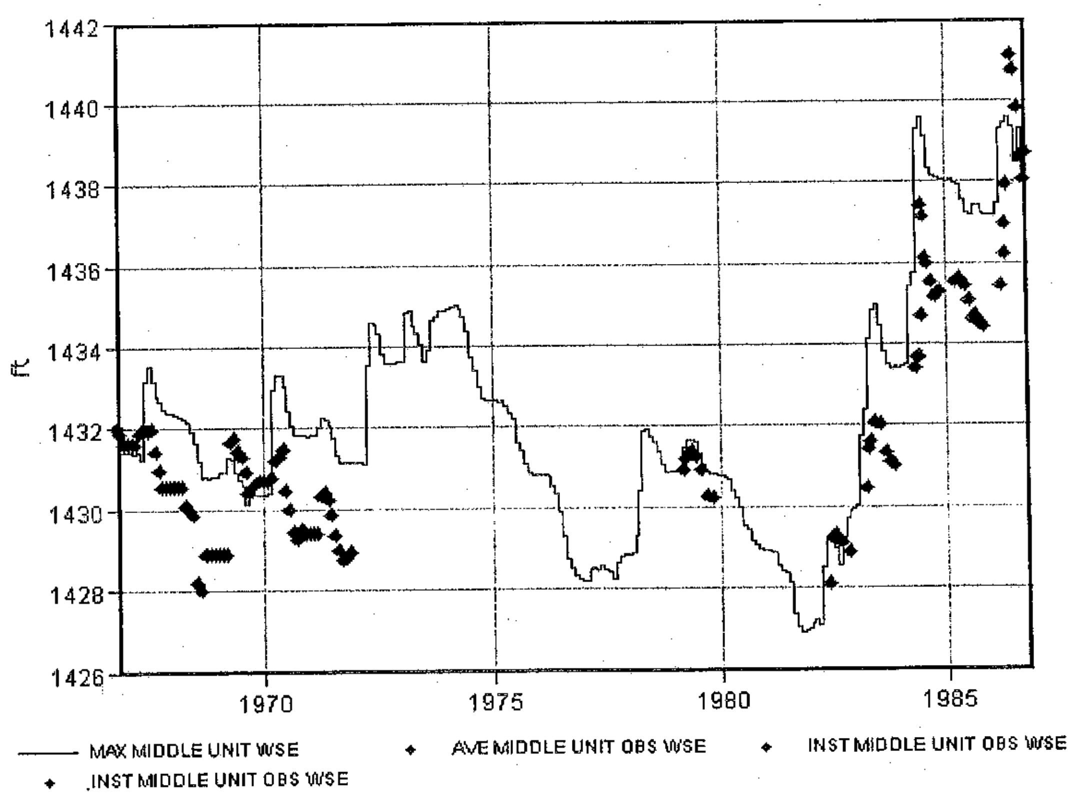

Figure B.6 Lake Andes Middle Unit simulated maximum water levels and observed water surface elevations from 1967 through 1986.

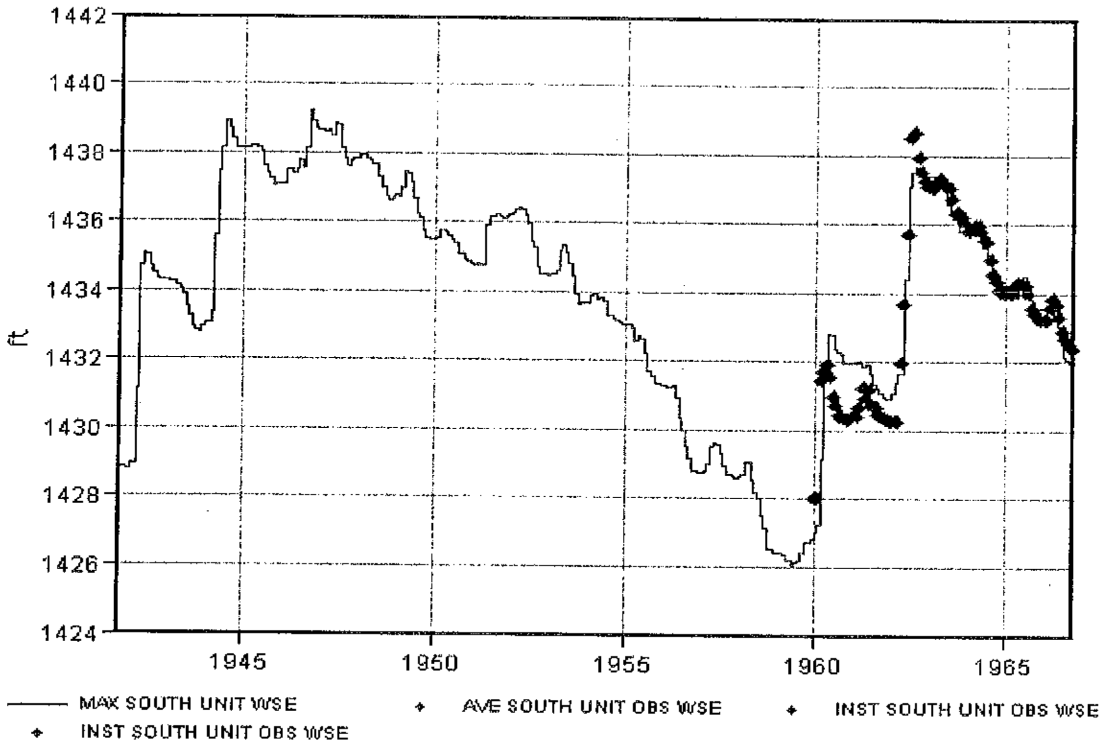

Figure B.7 Lake Andes South Unit simulated maximum water levels and observed water surface elevations, from 1942 through 1966.

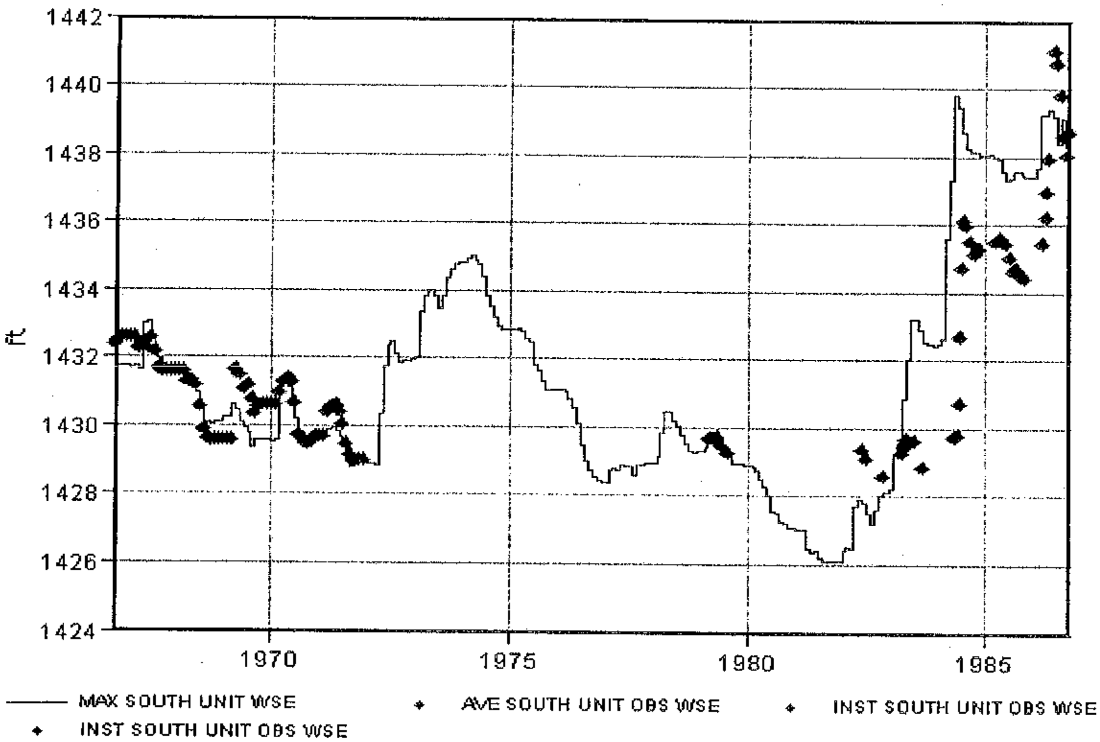

Figure B.8 Lake Andes South Unit simulated maximum water levels and observed water surface elevations from 1967 through 1986.

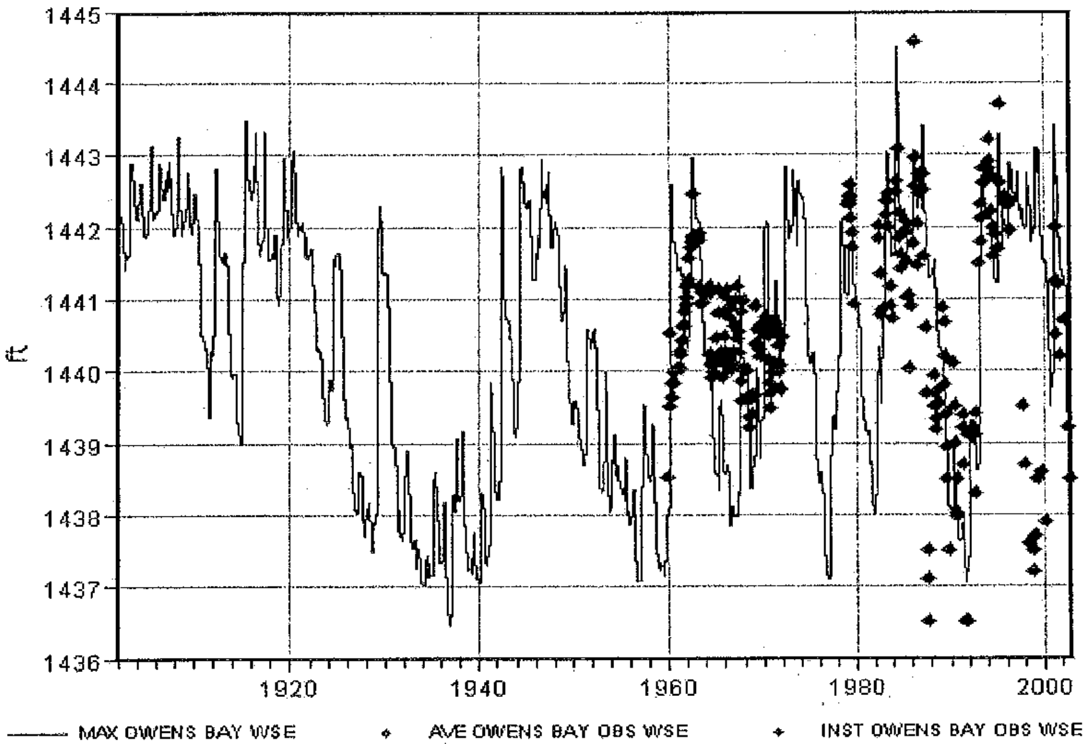

Figure B.9 Lake Andes Owen's Bay Unit simulated maximum water levels and observed water surface elevations from 1902 through 2001.

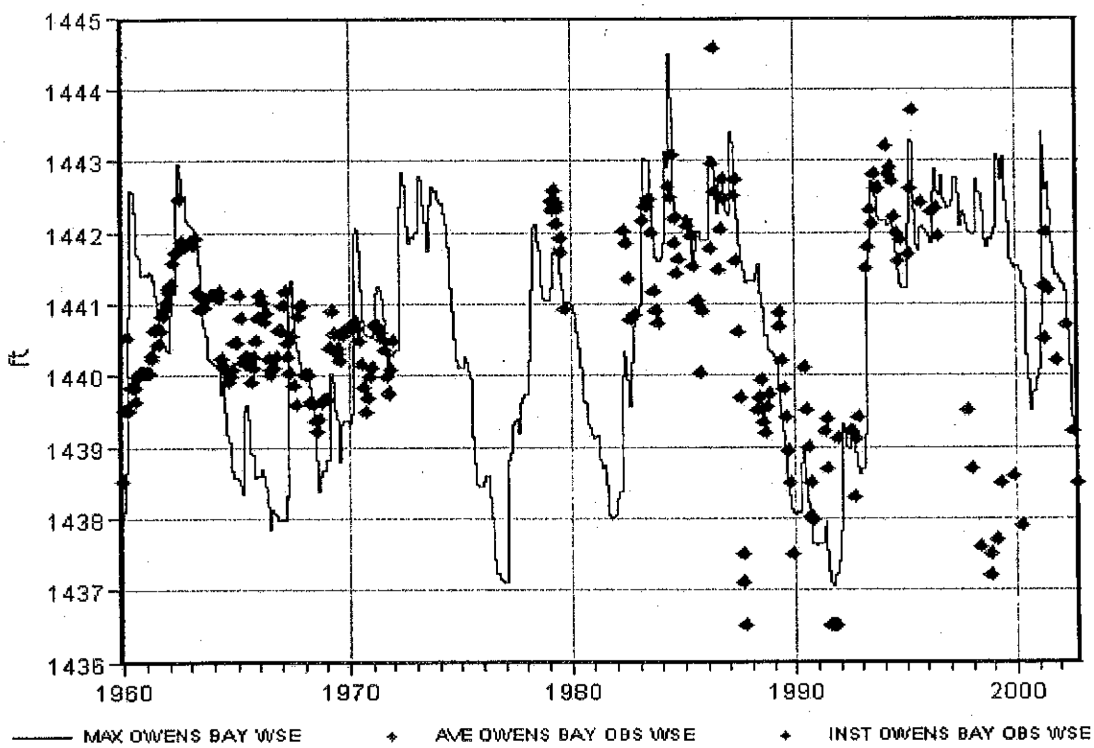

Figure B.10 Lake Andes Owen's Bay Unit simulated maximum water levels and observed water surface elevations from 1960 through 2002.

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TT Combined Lake (Units 1, 2, 3 & OWEN'S BAY)
TT
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CM      ISTART  = WATER YEAR TO START RUN
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CM      ISTORE  = 1 TO SAVE STATS IN FILE, 0 TO NOT
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CM              = 2 TO PUNCH DAILY STORAGE & AREA
CM              = 3 TO PUNCH DAILY WS ELEVATION & STG,AREA
CM              = 4 TO PUNCH DAILY AVE DEPTH & WSE,STG,AREA
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CM              = 0 TO NOT SAVE
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CM              = 0 TO CONTINUOUSLY SIMULATE WITH END STORAGE CARRYING
CM                  OVER TO INITIAL STORAGE
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LAKE ANDES BASELINE MODEL INPUT FILE

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CM      TOLPER   = TOLERANCE FOR CONVERGENCE IN PERCENT
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IC          36750
CM*************************************************************
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CM      AREA     = SURFACE AREA IN ACRES
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CM      QOUT     = OUTFLOW CAPACITY IN CFS
CM      DAVE     = AVERAGE DEPTH IN FEET
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CM     SMIN = SOIL MOISTURE INDEX WHICH IS COMPUTED IN SMI.FOR
CM     ROP  = RUNOFF PERCENT WHICH IS APPLIED TO PRECIPITATION
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LAKE ANDES BASELINE MODEL INPUT FILE

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CM      QFAC     = OUTFLOW FACTOR FOR MONTH IN PERCENT
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LAKE ANDES BASELINE MODEL INPUT FILE

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UG          .01
UG          .01
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FX        MAXTEMP.DAT
FN        MINTEMP.DAT
CM*********************************************************
ZZ
```

APPENDIX C. COINCIDENT FREQUENCY ANALYSIS

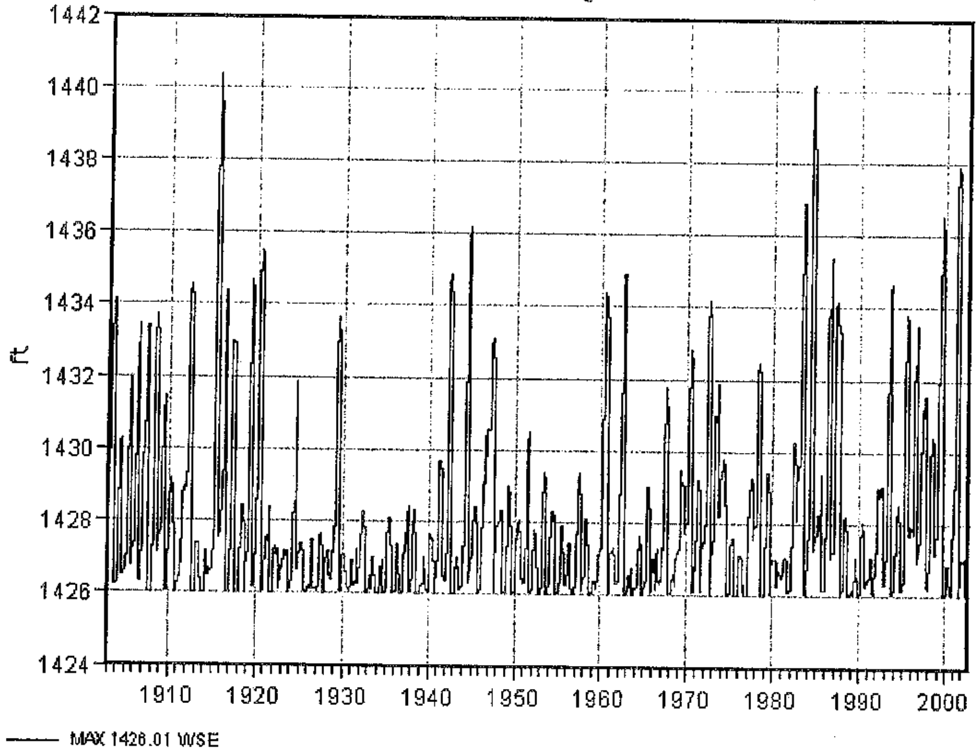

Figure C.1 Lake Andes Coincident Frequency Model, starting lake elevation = 1426.0 ft MSL.

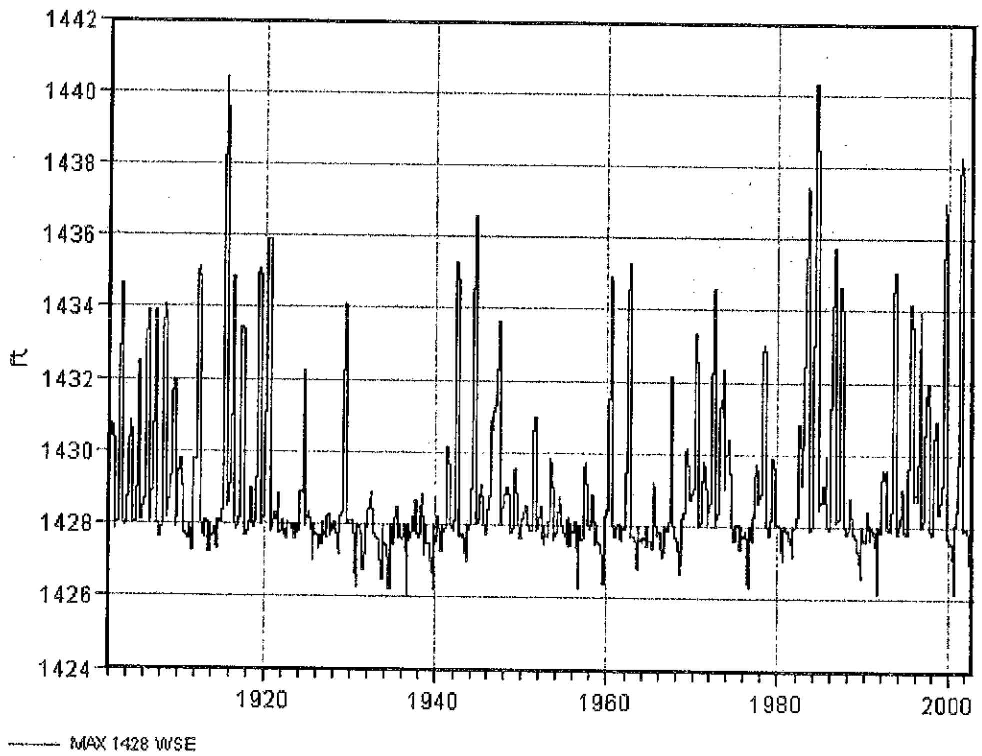

Figure C.2 Lake Andes Coincident Frequency Model, starting lake elevation = 1428.0 ft MSL.

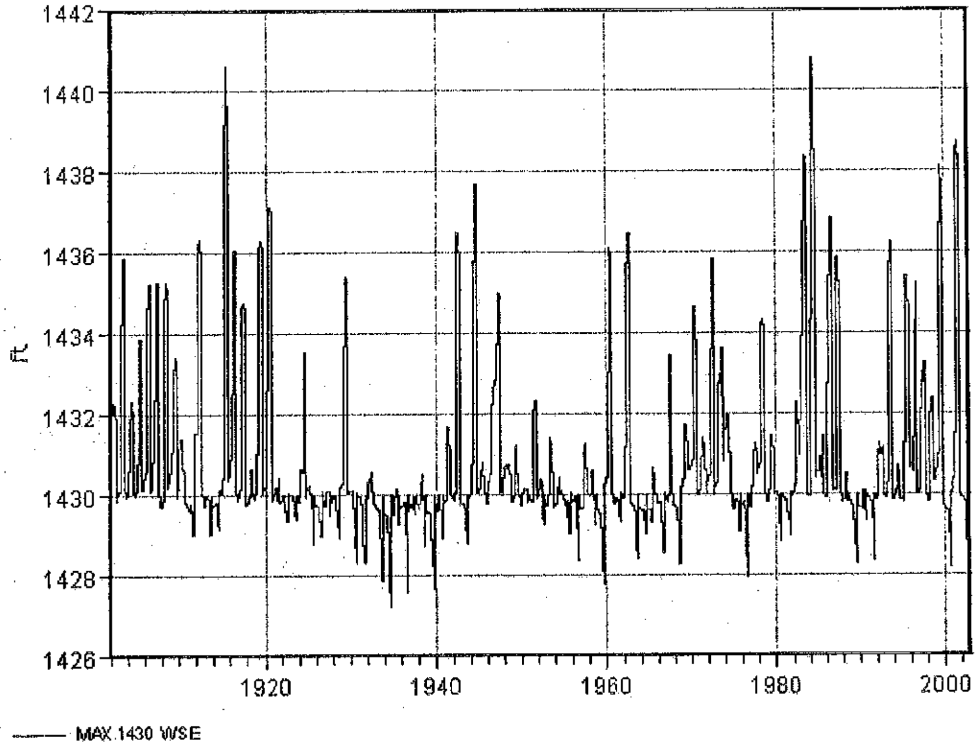

Figure C.3 Lake Andes Coincident Frequency Model, starting lake elevation = 1430.0 ft MSL.

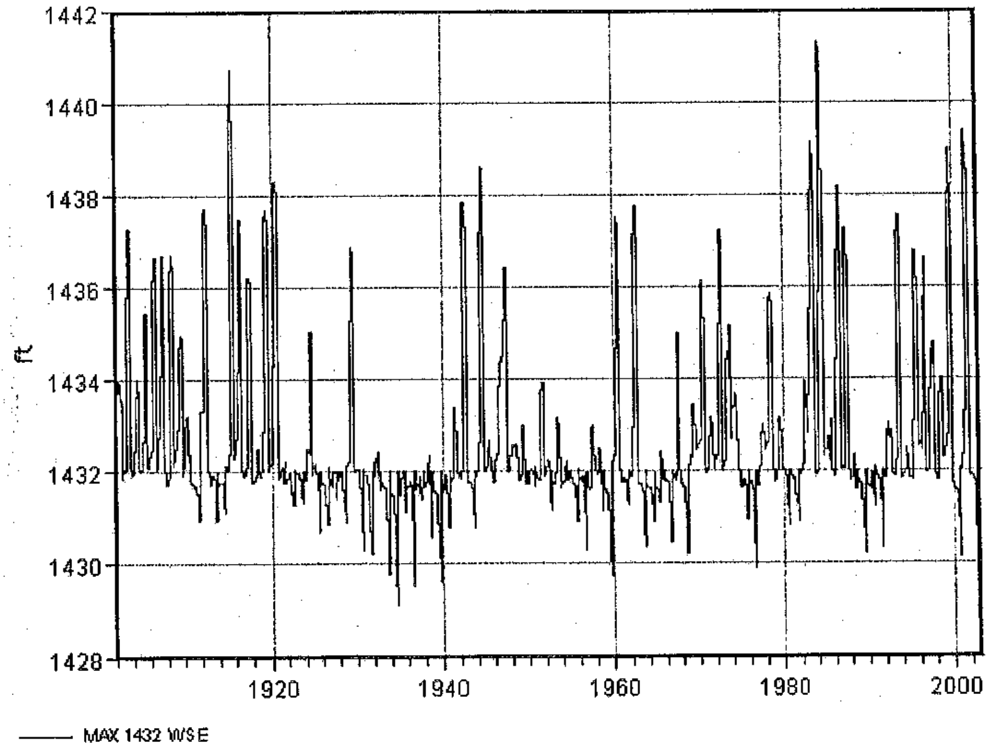

Figure C.4 Lake Andes Coincident Frequency Model, starting lake elevation = 1432.0 ft MSL.

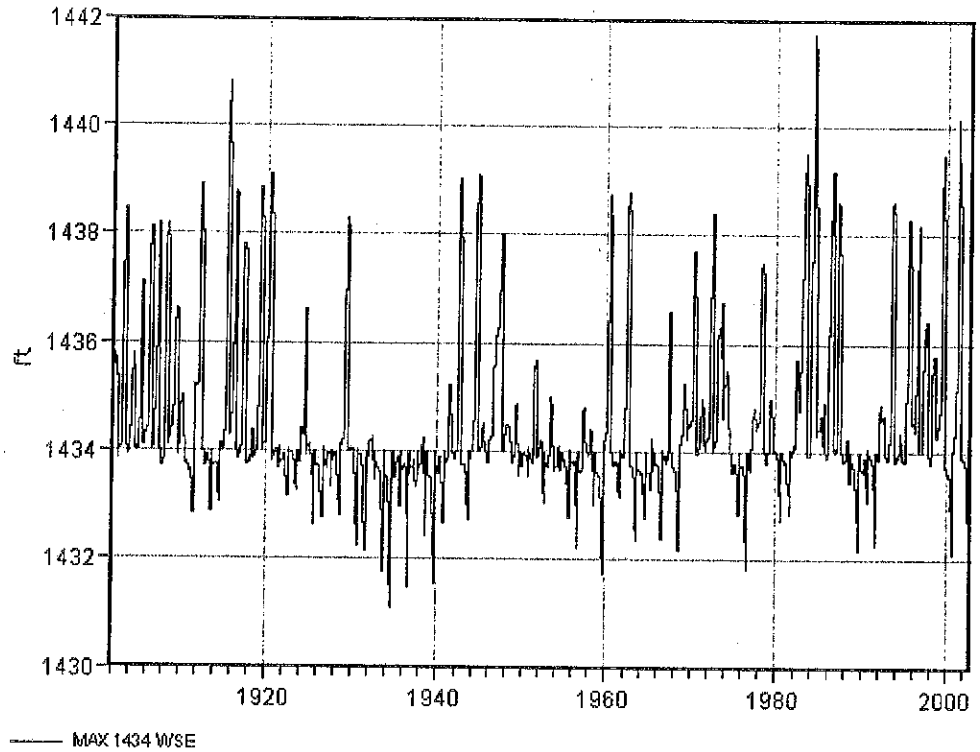

Figure C.5 Lake Andes Coincident Frequency Model, starting lake elevation = 1434.0 ft MSL.

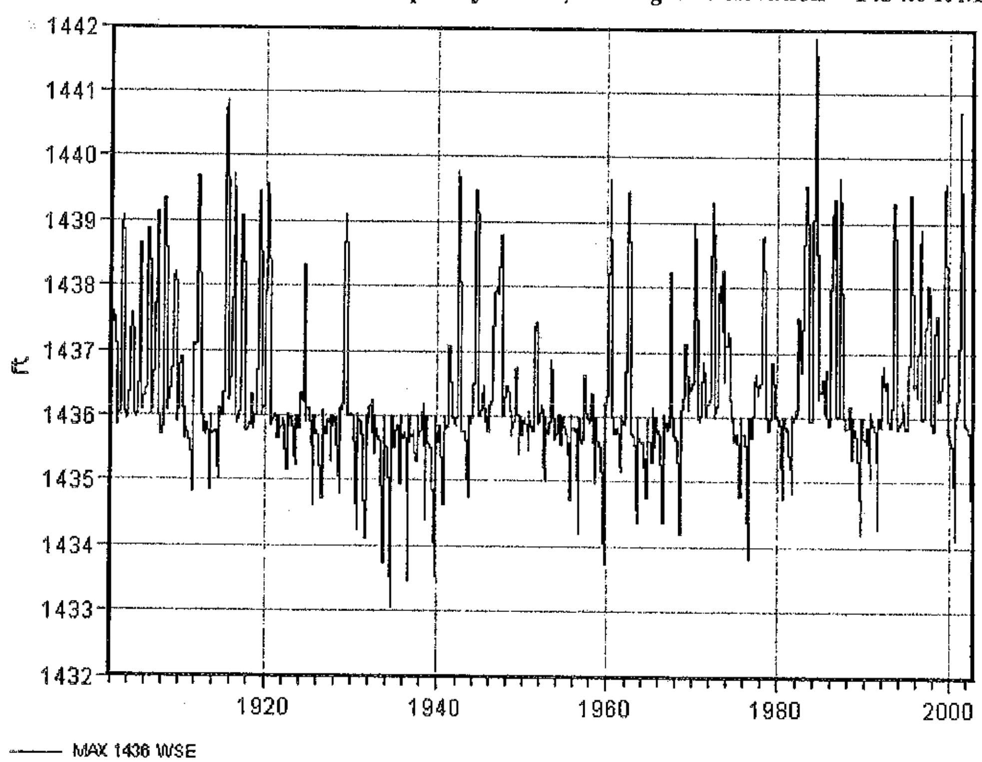

Figure C.6 Lake Andes Coincident Frequency Model, starting lake elevation = 1436.0 ft MSL.

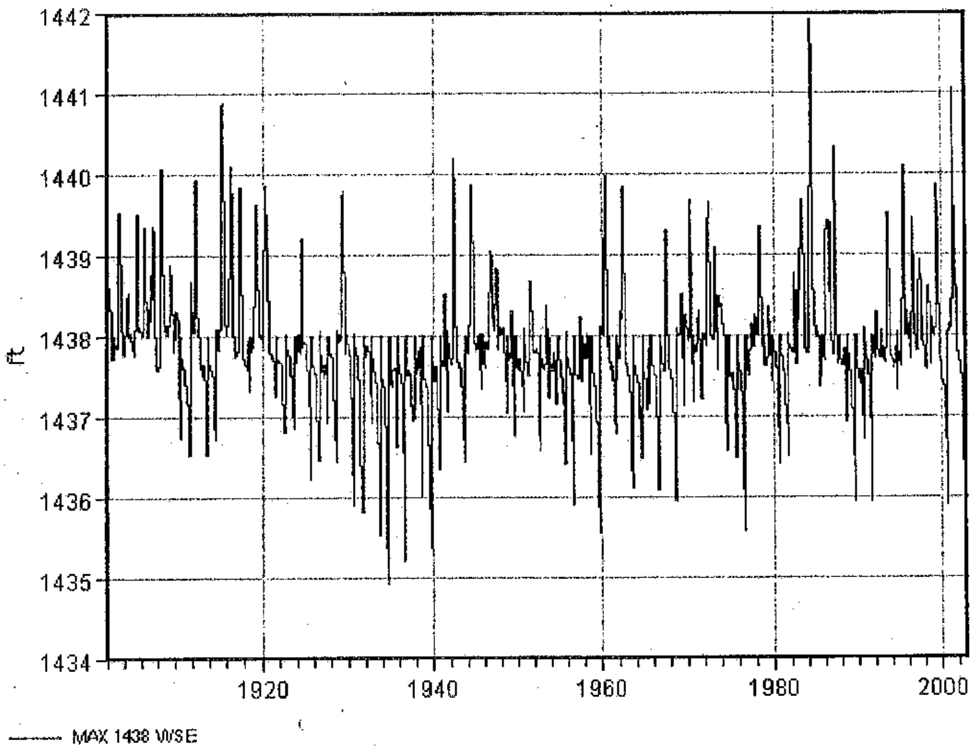

Figure C.7 Lake Andes Coincident Frequency Model, starting lake elevation = 1438.0 ft MSL.

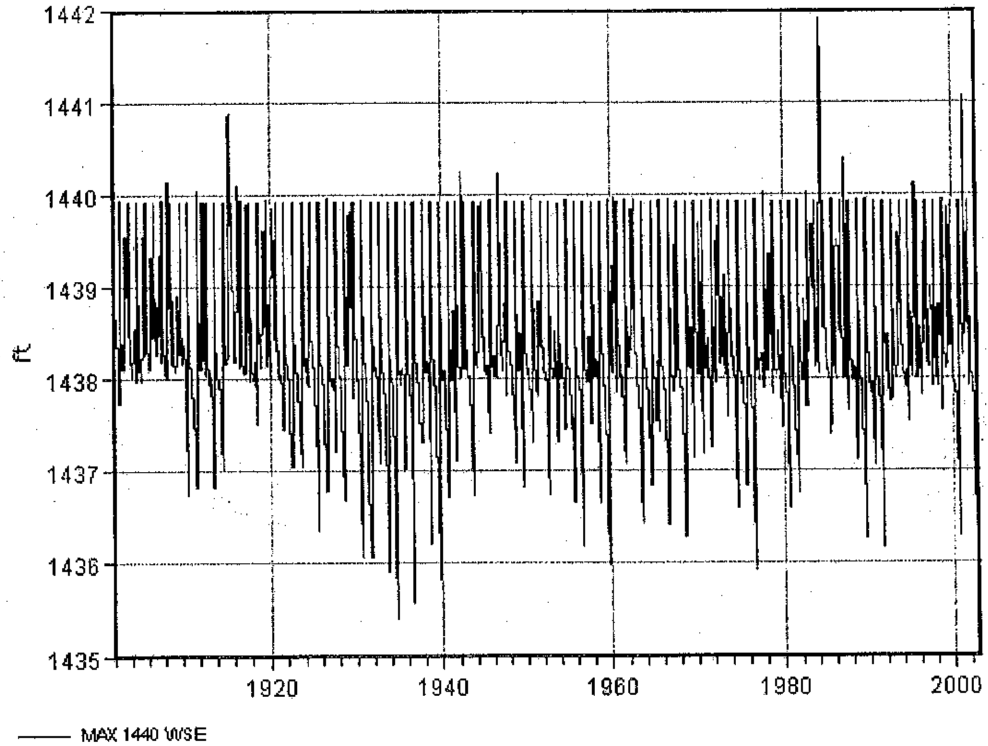

Figure C.8 Lake Andes Coincident Frequency Model, starting lake elevation = 1440.0 ft MSL.

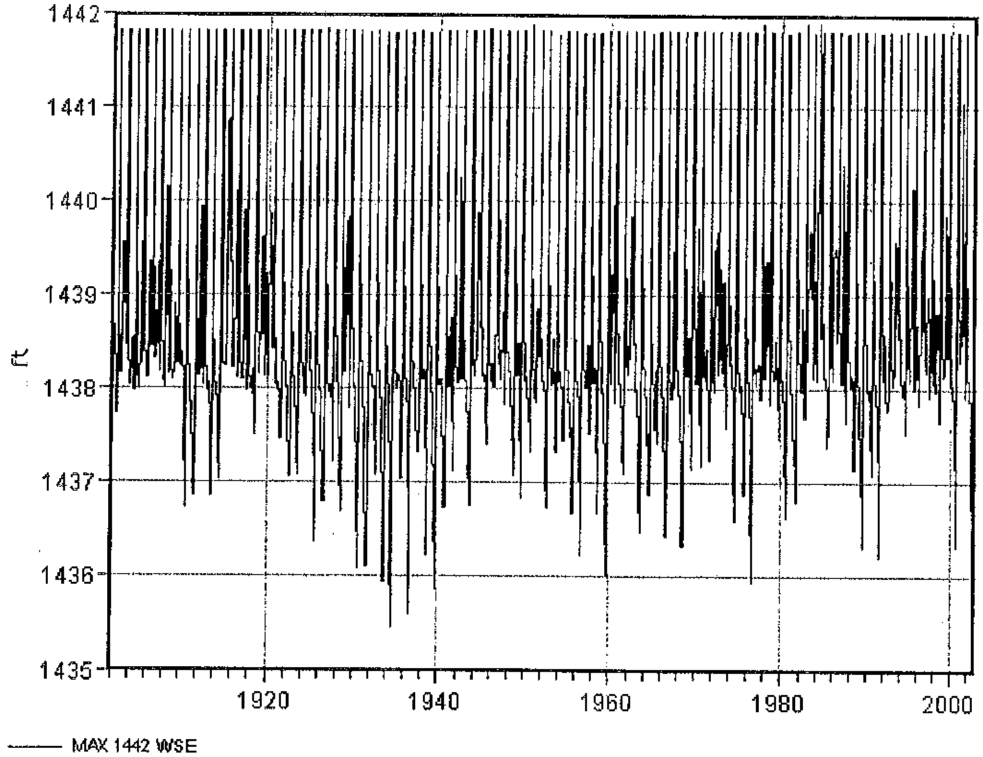

Figure C.9 Lake Andes Coincident Frequency Model, starting lake elevation = 1442.0 ft MSL.

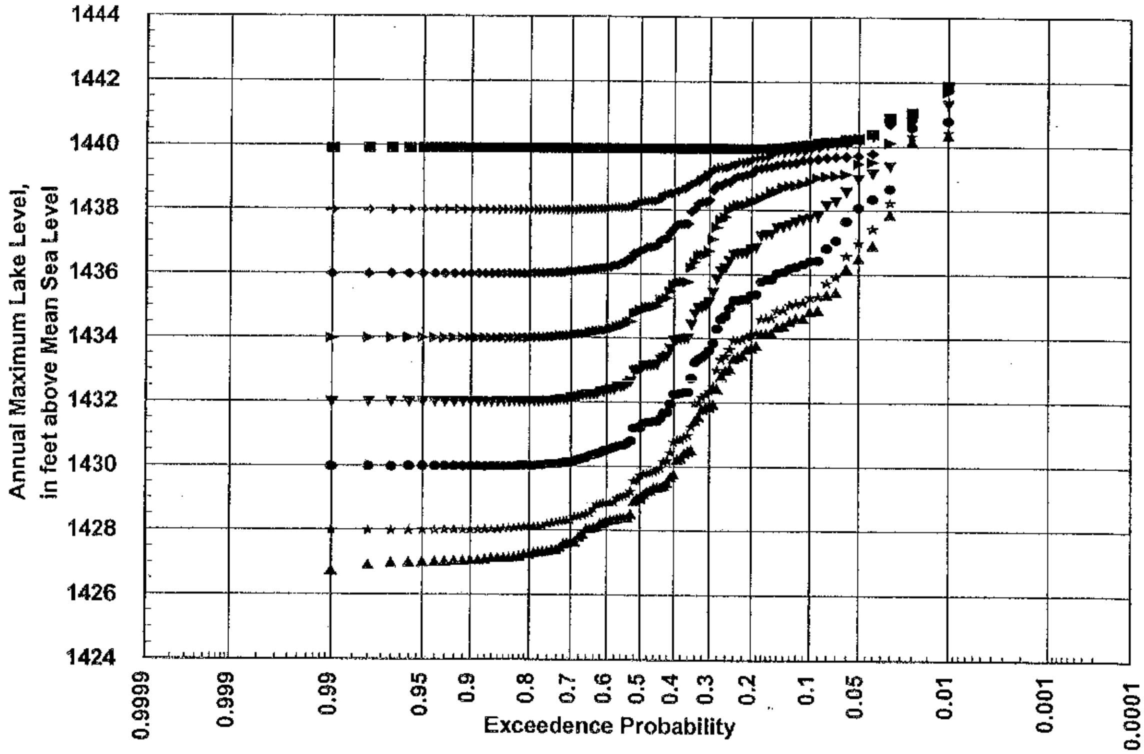

Figure C.10 Family of Weibull plotted frequencies curves for the coincident lake level frequency analysis. Each curve represents a simulation in which the starting lake level was reset to a prescribed elevation at the beginning of each water year.

APPENDIX D. WIND WAVE ANALYSIS

Table D.1 Lake Andes Wind-Wave Analysis Summary

Location	WS Elev. & Freq.	Dsn. Wind Mph.	Duration min	Period Sec.	Wave Ht. feet	Wave L. feet	2% Wave feet
North Dike, N. Side							
Wind Direction = N							
Fetch L.= 5000'	1433.25						
Depth = 2.5'	2-yr	37.5	12.0	1.8	1.2	16.6	1.7
(Shallow Water Condition)	10-yr	49.0	10.0	2.0	1.6	20.5	2.2
	100-yr	62.0	9.0	2.3	2.0	27.1	2.8
Fetch L.= 5900' = 1.12 mi	1437.25						
Depth = 6'	2-yr	37.5	13.5	1.9	1.3	18.5	1.8
(Shallow Water Condition)	10-yr	49.0	11.0	2.1	1.6	22.6	2.2
	100-yr	62.0	9.5	2.4	2.1	28.3	2.9
Fetch L= 6500' = 1.23 mi	1442.25						
Depth = 10'	2-yr	37.0	16.0	2.1	1.5	21.5	2.1
(Shallow Water Condition)	10-yr	48.5	13.0	2.3	2.0	27.1	2.8
	100-yr	61.5	11.0	2.5	2.6	32.0	3.6
North Dike, S. Side							
Wind Direction = SSW							
Fetch L=11300'	1433.25						
Depth = 5'	2-yr	33.0	23.0	2.2	1.3	24.8	1.8
(Shallow Water Condition)	10-yr	44.5	17.5	2.4	1.7	29.5	2.4
	100-yr	57.5	13.0	2.7	2.2	37.3	3.1
Fetch L=11900' =2.25 mi	1437.25						
Depth = 8'	2-yr	33.0	27.5	2.3	1.7	27.1	2.4
(Shallow Water Condition)	10-yr	44.0	22.0	2.7	2.3	37.3	3.2
	100-yr	56.5	18.0	2.9	3.0	43.1	4.2
Fetch L=12500' =2.37 mi	1442.25						
Depth = 12'	2-yr	32.5	28.0	2.3	1.7	27.1	2.4
(Shallow Water Condition)	10-yr	44.0	22.0	2.7	2.3	37.3	3.2
	100-yr	56.0	18.0	2.9	3.0	43.1	4.2
South Dike, N. Side							
Wind Direction = NNE							
Fetch L=10000'	1433.25						
Depth = 5'	2-yr	34.0	20.5	2.2	1.4	24.8	2.0
(Shallow Water Condition)	10-yr	44.0	16.5	2.4	1.7	29.5	2.4
	100-yr	57.0	13.5	2.6	2.1	34.6	2.9
Fetch L=10550' =2 mi	1437.25						
Depth = 8'	2-yr	34.0	24.0	2.3	1.6	25.9	2.2
(Shallow Water Condition)	10-yr	44.0	19.5	2.5	2.2	32.0	3.1
	100-yr	57.0	15.5	2.8	2.8	40.1	3.9
Fetch L=11600' =2.2 mi	1442.25						
Depth = 12'	2-yr	34.0	25.5	2.3	1.7	27.1	2.4
(Shallow Water Condition)	10-yr	43.0	21.0	2.6	2.2	34.6	3.1

Location	WS Elev. & Freq.	Dsn. Wind Mph.	Duration min	Period Sec.	Wave Ht. feet	Wave L. feet	2% Wave feet
	100-yr	57.0	17.0	2.8	2.9	40.1	4.1
South Dike, S. Side							
Wind Direction = S							
Fetch L=4275'	1433.25						
Depth = 6'	2-yr	34.0	11.5	1.7	1.0	14.8	1.4
(Shallow Water Condition)	10-yr	45.0	10.0	1.9	1.4	18.5	2.0
	100-yr	59.0	8.0	2.1	1.8	22.6	2.5
Fetch L=4500'=.85 mi	1437.25						
Depth = 10'	2-yr	33.0	17.5	1.8	1.2	16.6	1.7
(Deep Water Condition)	10-yr	44.5	13.0	2.1	1.8	21.5	2.5
	100-yr	57.5	10.0	2.3	2.4	27.1	3.4
Fetch L=5020'=.95 mi	1442.25						
Depth = 15'	2-yr	33.0	23.0	2.0	1.3	19.5	1.8
(Deep Water Condition)	10-yr	44.0	18.0	2.2	1.9	24.8	2.6
	100-yr	56.0	15.0	2.4	2.5	29.5	3.5
Owen's Bay Emb, W. Side							
Wind Direction = W							
Fetch L=11730'	1433.25						
Depth = 6'	2-yr	34.0	21.0	2.2	1.3	24.8	1.8
(Shallow Water Condition)	10-yr	48.0	18.0	2.5	1.8	32.0	2.5
	100-yr	64.5	15.0	2.7	2.3	37.3	3.2
Wind Direction = W							
Fetch L=13800' = 2.61 mi	1437.25						
Depth = 10'	2-yr	34.0	27.0	2.4	1.8	29.5	2.5
(Shallow Water Condition)	10-yr	48.0	21.5	2.8	2.7	40.1	3.8
	100-yr	64.0	17.5	3.1	3.4	49.2	4.8
Fetch L=15870' = 3 mi	1442.25						
Depth = 15'	2-yr	34.0	33.0	2.6	2.0	34.6	2.8
(Shallow Water Condition)	10-yr	47.5	24.5	3.0	3.0	46.1	4.2
	100-yr	64.0	21.5	3.3	4.0	55.8	5.6
Hwy 281, N. Side							
Wind Direction = NNE							
Fetch L=4050'	1433.25						
Depth = 6'	2-yr	35.0	11.5	1.7	1.1	14.8	1.5
(Shallow Water Condition)	10-yr	46.0	9.0	1.9	1.4	18.5	2.0
	100-yr	59.0	7.5	2.1	1.8	22.6	2.5
Fetch L=4500'=.85 mi	1437.25						
Depth = 10'	2-yr	35.0	16.0	1.9	1.3	17.5	1.8
(Deep Water Condition)	10-yr	45.0	11.0	2.1	1.8	22.6	2.5
	100-yr	58.0	8.0	2.4	2.5	29.5	3.5
Fetch L=4720'=.89 mi	1442.25						
Depth = 15'	2-yr	35.0	16.0	1.9	1.3	18.5	1.8

Location	WS Elev. & Freq.	Dsn. Wind Mph.	Duration min	Period Sec.	Wave Ht. feet	Wave L. feet	2% Wave feet
(Deep Water Condition)	10-yr	45.0	11.0	2.1	1.8	23.4	2.5
	100-yr	57.5	10.0	2.5	2.5	30.7	3.5

Table D.2 Lake Andes Wind-Wave Design Runup and Setup

Location	WS Elev. & Freq.	Dsn. Wind Mph.	Period Sec.	Wave Ht. feet	Wave L. feet	Setup feet	Run-up feet
North Dike, N. Side							
Wind Direction = N							
Embankment Side Slope	3						
Fetch L.= 5000'	1433.25						
Depth = 2.5'	2-yr	37.5	1.8	1.2	16.6	0.8	1.5
(Shallow Water Condition)	10-yr	49.0	2.0	1.6	20.5	1.3	1.9
	100-yr	62.0	2.3	2.0	27.1	2.1	2.5
Fetch L.= 5900' = 1.12 mi	1437.25						
Depth = 6'	2-yr	37.5	1.9	1.3	18.5	0.4	1.6
(Shallow Water Condition)	10-yr	49.0	2.1	1.6	22.6	0.6	2.0
	100-yr	62.0	2.4	2.1	28.3	1.0	2.6
Fetch L.= 6500' = 1.23 mi	1442.25						
Depth = 10'	2-yr	37.0	2.1	1.5	21.5	0.2	1.9
(Shallow Water Condition)	10-yr	48.5	2.3	2.0	27.1	0.4	2.5
	100-yr	61.5	2.5	2.6	32.0	0.7	3.1
North Dike, S. Side							
Wind Direction = SSW							
Embankment Side Slope	3						
Fetch L=11300'	1433.25						
Depth = 5'	2-yr	33.0	2.2	1.3	24.8	0.7	1.8
(Shallow Water Condition)	10-yr	44.5	2.4	1.7	29.5	1.2	2.3
	100-yr	57.5	2.7	2.2	37.3	2.0	2.9
Fetch L=11900' =2.25 mi	1437.25						
Depth = 8'	2-yr	33.0	2.3	1.7	27.1	0.4	2.2
(Shallow Water Condition)	10-yr	44.0	2.7	2.3	37.3	0.8	3.0
	100-yr	56.5	2.9	3.0	43.1	1.3	3.8
Fetch L=12500' =2.37 mi	1442.25						
Depth = 12'	2-yr	32.5	2.3	1.7	27.1	0.3	2.2
(Shallow Water Condition)	10-yr	44.0	2.7	2.3	37.3	0.5	3.0
	100-yr	56.0	2.9	3.0	43.1	0.9	3.8
South Dike, N. Side							
Wind Direction = NNE							
Embankment Side Slope	4						
Fetch L=10000'	1433.25						
Depth = 5'	2-yr	34.0	2.2	1.4	24.8	0.6	1.9
(Shallow Water Condition)	10-yr	44.0	2.4	1.7	29.5	1.0	2.3
	100-yr	57.0	2.6	2.1	34.6	1.8	2.8
Fetch L=10550' =2 mi	1437.25						

Location	WS Elev. & Freq.	Dsn. Wind Mph.	Period Sec.	Wave Ht. feet	Wave L. feet	Setup feet	Run-up feet
Depth = 8'	2-yr	34.0	2.3	1.6	25.9	0.4	1.7
(Shallow Water Condition)	10-yr	44.0	2.5	2.2	32.0	0.7	2.3
	100-yr	57.0	2.8	2.8	40.1	1.2	2.9
Fetch L=11600' =2.2 mi	1442.25						
Depth = 12'	2-yr	34.0	2.3	1.7	27.1	0.3	1.8
(Shallow Water Condition)	10-yr	43.0	2.6	2.2	34.6	0.5	2.3
	100-yr	57.0	2.8	2.9	40.1	0.9	2.9
Location							
South Dike, S. Side							
Wind Direction = S							
Embankment Side Slope	4						
Fetch L=4275'	1433.25						
Depth = 6'	2-yr	34.0	1.7	1.0	14.8	0.2	1.3
(Shallow Water Condition)	10-yr	45.0	1.9	1.4	18.5	0.4	1.7
	100-yr	59.0	2.1	1.8	22.6	0.7	2.2
Fetch L=4500'=.85 mi	1437.25						
Depth = 10'	2-yr	33.0	1.8	1.2	16.6	0.1	1.2
(Deep Water Condition)	10-yr	44.5	2.1	1.8	21.5	0.2	1.7
	100-yr	57.5	2.3	2.4	27.1	0.4	2.3
Fetch L=5020'=.95 mi	1442.25						
Depth = 15'	2-yr	33.0	2.0	1.3	19.5	0.1	1.4
(Deep Water Condition)	10-yr	44.0	2.2	1.9	24.8	0.2	1.9
	100-yr	56.0	2.4	2.5	29.5	0.3	2.4
Owens Bay Emb, W. Side							
Wind Direction = W							
Embankment Side Slope	2.5						
Fetch L=11730'	1433.25						
Depth = 6'	2-yr	34.0	2.2	1.3	24.8	0.6	1.8
(Shallow Water Condition)	10-yr	48.0	2.5	1.8	32.0	1.2	2.4
	100-yr	64.5	2.7	2.3	37.3	2.2	3.0
Fetch L=13800' = 2.61 mi	1437.25						
Depth = 10'	2-yr	34.0	2.4	1.8	29.5	0.4	2.7
(Shallow Water Condition)	10-yr	48.0	2.8	2.7	40.1	0.9	3.9
	100-yr	64.0	3.1	3.4	49.2	1.5	4.8
Fetch L=15870' = 3 mi	1442.25						
Depth = 15'	2-yr	34.0	2.6	2.0	34.6	0.3	3.0
(Shallow Water Condition)	10-yr	47.5	3.0	3.0	46.1	0.6	4.3
	100-yr	64.0	3.3	4.0	55.8	1.2	5.6
Hwy 281, N. Side							
Wind Direction = NNE							
Embankment Side Slope	3						
Fetch L=4050'	1433.25						

Location	WS Elev. & Freq.	Dsn. Wind Mph.	Period Sec.	Wave Ht. feet	Wave L. feet	Setup feet	Run-up feet
Depth = 6'	2-yr	35.0	1.7	1.1	14.8	0.2	1.4
(Shallow Water Condition)	10-yr	46.0	1.9	1.4	18.5	0.4	1.7
	100-yr	59.0	2.1	1.8	22.6	0.6	2.2
Fetch L=4500'=.85 mi	1437.25						
Depth = 10'	2-yr	35.0	1.9	1.3	17.5	0.1	1.6
(Deep Water Condition)	10-yr	45.0	2.1	1.8	22.6	0.2	2.1
	100-yr	58.0	2.4	2.5	29.5	0.4	2.9
Fetch L=4720'=.89 mi	1442.25						
Depth = 15'	2-yr	35.0	1.9	1.3	18.5	0.1	1.6
(Deep Water Condition)	10-yr	45.0	2.1	1.8	23.4	0.2	2.2
	100-yr	57.5	2.5	2.5	30.7	0.3	3.0

Table D.3 Lake Andes Wind-Wave Design Elevations

Location	WS Elev. & Freq.	Dsn. Wind Mph.	Setup feet	Wave Ht. feet	Design Run-up feet	Design Elevation Feet msl
North Dike, N. Side						
Wind Direction = N						
Embankment Side Slope	3					
Fetch L.= 5000'	1433.25					
Depth = 2.5'	2-yr	37.5	0.8	1.2	1.5	1435.5
(Shallow Water Condition)	10-yr	49.0	1.3	1.6	1.9	1436.5
	100-yr	62.0	2.1	2.0	2.5	1437.8
Fetch L.= 5900' = 1.12 mi	1437.25					
Depth = 6'	2-yr	37.5	0.4	1.3	1.6	1439.3
(Shallow Water Condition)	10-yr	49.0	0.6	1.6	2.0	1439.9
	100-yr	62.0	1.0	2.1	2.6	1440.9
Fetch L.= 6500' = 1.23 mi	1442.25					
Depth = 10'	2-yr	37.0	0.2	1.5	1.9	1444.4
(Shallow Water Condition)	10-yr	48.5	0.4	2.0	2.5	1445.1
	100-yr	61.5	0.7	2.6	3.1	1446.0
North Dike, S. Side						
Wind Direction = SSW						
Embankment Side Slope	3					
Fetch L=11300'	1433.25					
Depth = 5'	2-yr	33.0	0.7	1.3	1.8	1435.7
(Shallow Water Condition)	10-yr	44.5	1.2	1.7	2.3	1436.7
	100-yr	57.5	2.0	2.2	2.9	1438.2
Fetch L=11900' =2.25 mi	1437.25					
Depth = 8'	2-yr	33.0	0.4	1.7	2.2	1439.9
(Shallow Water Condition)	10-yr	44.0	0.8	2.3	3.0	1441.0
	100-yr	56.5	1.3	3.0	3.8	1442.3
Fetch L=12500' =2.37 mi	1442.25					

Location	WS Elev. & Freq.	Dsn. Wind Mph.	Setup feet	Wave Ht. feet	Design Run-up feet	Design Elevation Feet msl
Depth = 12'	2-yr	32.5	0.3	1.7	2.2	1444.8
(Shallow Water Condition)	10-yr	44.0	0.5	2.3	3.0	1445.8
	100-yr	56.0	0.9	3.0	3.8	1446.9
South Dike, N. Side						
Wind Direction = NNE						
Embankment Side Slope	4					
Fetch L=10000'	1433.25					
Depth = 5'	2-yr	34.0	0.6	1.4	1.9	1435.8
(Shallow Water Condition)	10-yr	44.0	1.0	1.7	2.3	1436.6
	100-yr	57.0	1.8	2.1	2.8	1437.8
Fetch L=10550' =2 mi	1437.25					
Depth = 8'	2-yr	34.0	0.4	1.6	1.7	1439.4
(Shallow Water Condition)	10-yr	44.0	0.7	2.2	2.3	1440.2
	100-yr	57.0	1.2	2.8	2.9	1441.3
Fetch L=11600' =2.2 mi	1442.25					
Depth = 12'	2-yr	34.0	0.3	1.7	1.8	1444.4
(Shallow Water Condition)	10-yr	43.0	0.5	2.2	2.3	1445.1
	100-yr	57.0	0.9	2.9	2.9	1446.0
Location	WS Elev.	Dsn. Wind	Setup	Wave Ht.	Run-up	Elevation
South Dike, S. Side	& Freq.	mph.	feet	feet	feet	feet msl
Wind Direction = S						
Embankment Side Slope	4					
Fetch L=4275'	1433.25					
Depth = 6'	2-yr	34.0	0.2	1.0	1.3	1434.7
(Shallow Water Condition)	10-yr	45.0	0.4	1.4	1.7	1435.4
	100-yr	59.0	0.7	1.8	2.2	1436.1
Fetch L=4500'=.85 mi	1437.25					
Depth = 10'	2-yr	33.0	0.1	1.2	1.2	1438.6
(Deep Water Condition)	10-yr	44.5	0.2	1.8	1.7	1439.2
	100-yr	57.5	0.4	2.4	2.3	1439.9
Fetch L=5020'=.95 mi	1442.25					
Depth = 15'	2-yr	33.0	0.1	1.3	1.4	1443.7
(Deep Water Condition)	10-yr	44.0	0.2	1.9	1.9	1444.3
	100-yr	56.0	0.3	2.5	2.4	1444.9
Owens Bay Emb, W. Side						
Wind Direction = W						
Embankment Side Slope	2.5					
Fetch L=11730'	1433.25					
Depth = 6'	2-yr	34.0	0.6	1.3	1.8	1435.7
(Shallow Water Condition)	10-yr	48.0	1.2	1.8	2.4	1436.9
	100-yr	64.5	2.2	2.3	3.0	1438.5
Fetch L=13800' = 2.61 mi	1437.25					
Depth = 10'	2-yr	34.0	0.4	1.8	2.7	1440.3

Location	WS Elev. & Freq.	Dsn. Wind Mph.	Setup feet	Wave Ht. feet	Design Run-up feet	Design Elevation Feet msl
(Shallow Water Condition)	10-yr	48.0	0.9	2.7	3.9	1442.0
	100-yr	64.0	1.5	3.4	4.8	1443.6
Fetch L=15870' = 3 mi	1442.25					
Depth = 15'	2-yr	34.0	0.3	2.0	3.0	1445.6
(Shallow Water Condition)	10-yr	47.5	0.6	3.0	4.3	1447.2
	100-yr	64.0	1.2	4.0	5.6	1449.0
Hwy 281, N. Side						
Wind Direction = NNE						
Embankment Side Slope	3					
Fetch L=4050'	1433.25					
Depth = 6'	2-yr	35.0	0.2	1.1	1.4	1434.8
(Shallow Water Condition)	10-yr	46.0	0.4	1.4	1.7	1435.3
	100-yr	59.0	0.6	1.8	2.2	1436.1
Fetch L=4500'=.85 mi	1437.25					
Depth = 10'	2-yr	35.0	0.1	1.3	1.6	1439.0
(Deep Water Condition)	10-yr	45.0	0.2	1.8	2.1	1439.6
	100-yr	58.0	0.4	2.5	2.9	1440.6
Fetch L=4720'=.89 mi	1442.25					
Depth = 15'	2-yr	35.0	0.1	1.3	1.6	1444.0
(Deep Water Condition)	10-yr	45.0	0.2	1.8	2.2	1444.6
	100-yr	57.5	0.3	2.5	3.0	1445.5

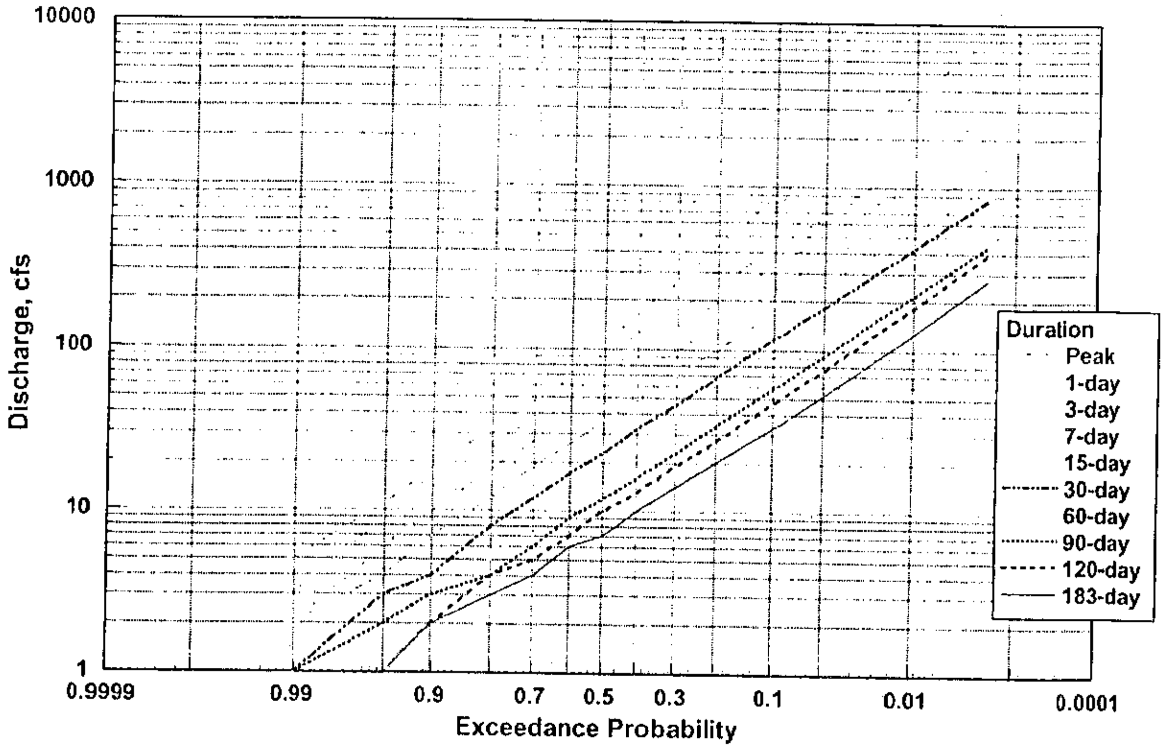

Figure E.1 Lake Andes North Unit discharge volume probability curves.

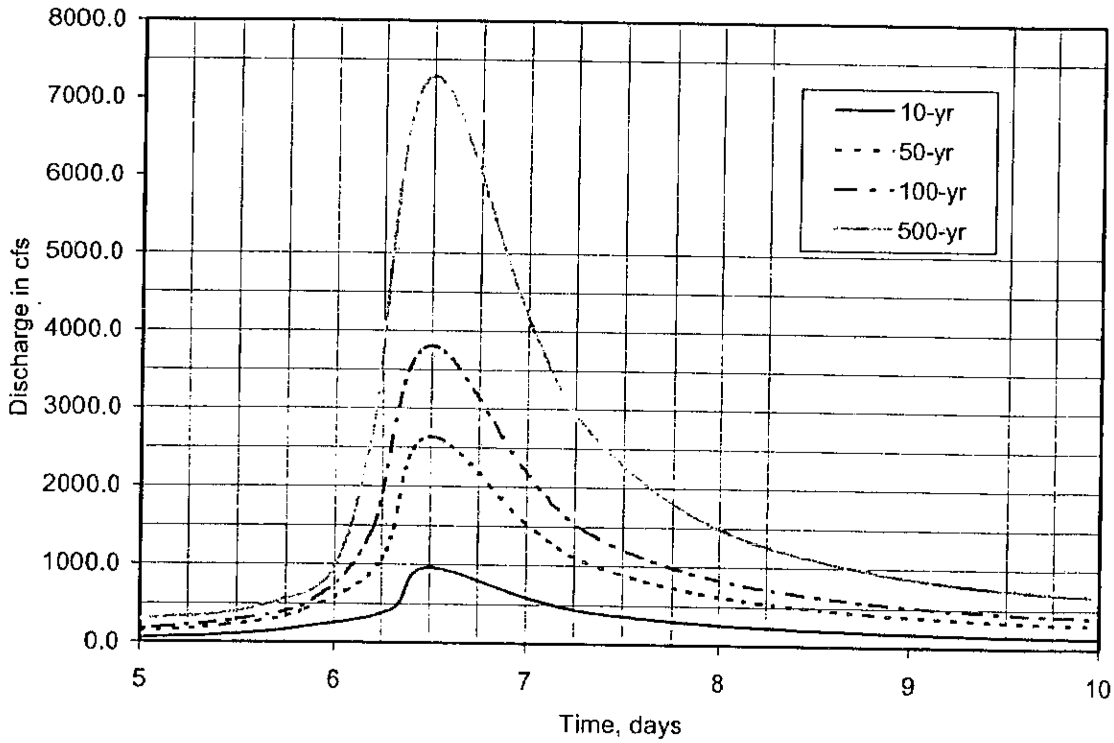

Figure E.2 Balanced flood hydrographs generated from the North Unit volume-probability curves.

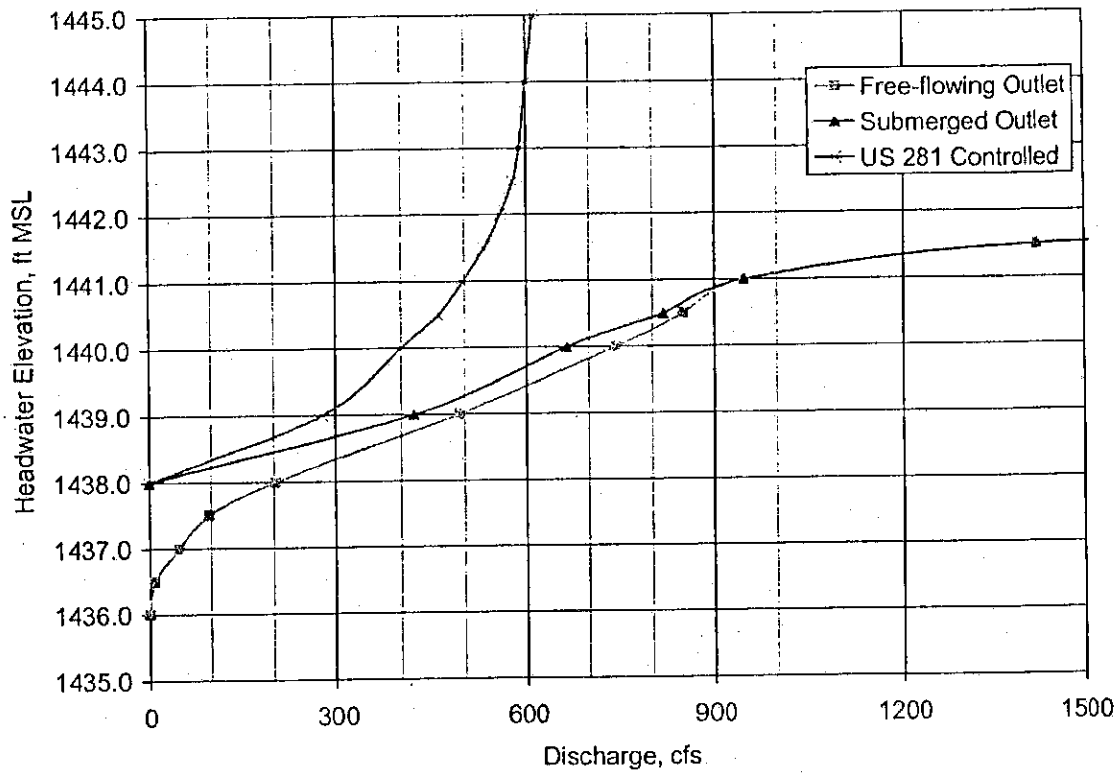

Figure E.3 North Unit free-flowing, submerged, and US281 controlled outflow curves for the HEC-HMS flood routing analysis.

Table E.1 North Pool HEC-HMS flood routing analysis model inputs.

Drainage Area (mi^2) = 72.85				
		----- Middle Pool Elevation -----		
		< 1437.25	1438.25	1441.00
		--------- Outlet Condition ---------		
---- North Pool ----		Free Flowing	Partially Submerged	US 281 Limited
Elevation	Storage	------------ Outflow ------------		
ft MSL	ac ft	cfs	cfs	cfs
1429.0	0	0	0	0
1432.0	250	0	0	0
1433.0	530	0	0	0
1434.0	920	0	0	0
1435.0	1430	0	0	0
1436.0	1950	0	0	0
1436.5	2200	8	0	0
1437.0	2540	48	0	0
1437.5	2850	97	0	0
1438.0	3170	203	0	0
1439.0	3830	492	421	278
1440.0	4470	742	665	400
1440.5	4830	852	819	458
1441.0	5200	948	948	498
1441.5	5550	1420	1420	532
1442.0	5900	3623	3623	558
1442.5	6250	8431	8431	578
1443.0	6600	15386	15386	588
1444.0	7300	35322	35322	600
1445.0	8000			612
Starting North Pool Elevations				
50 % duration pool			1435.8	ft MSL
2% (50-year) probability pool			1441.0	ft MSL

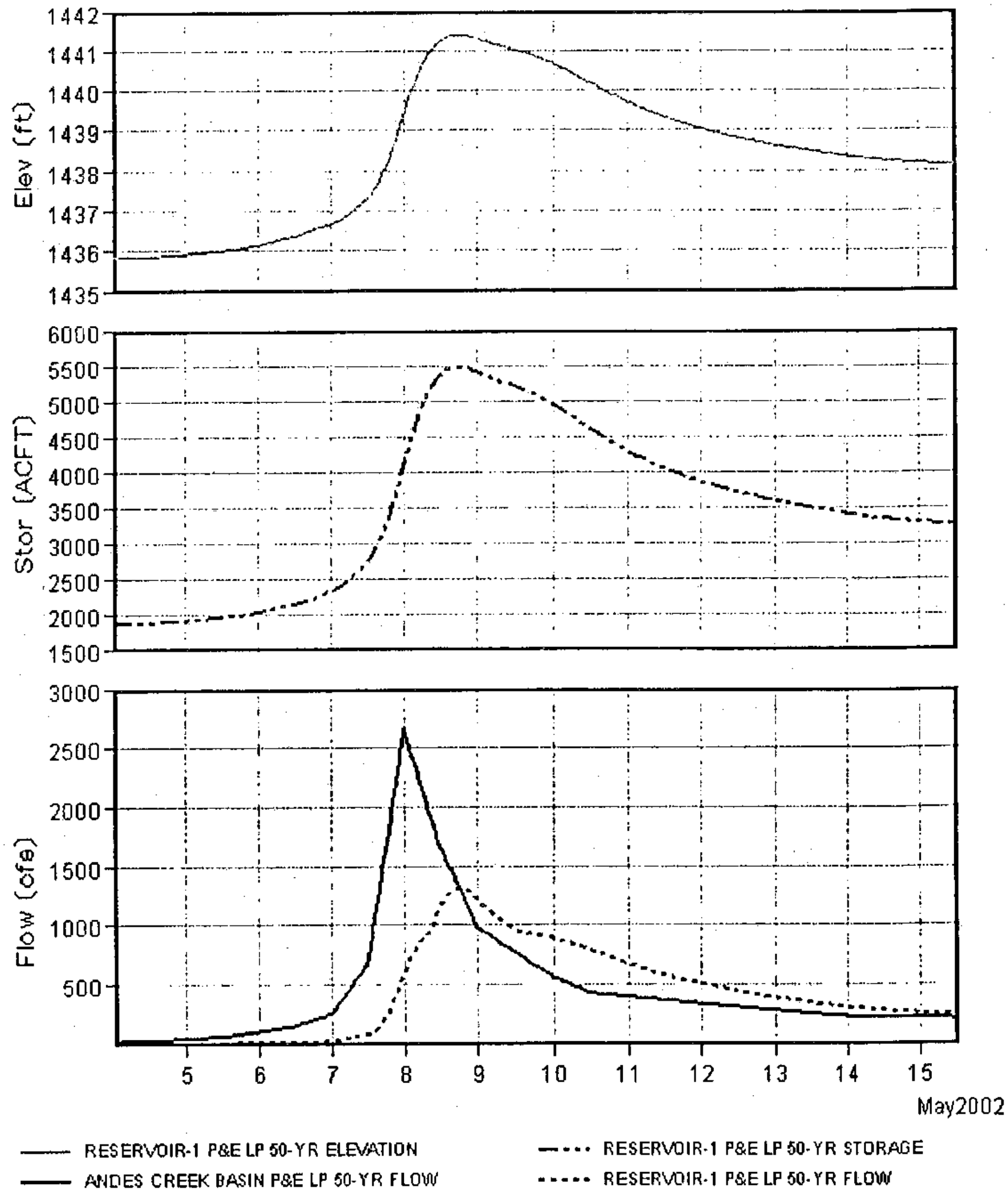

Figure E.4 Lake Andes North Unit HEC-HMS flood routing analysis: Starting pool elevation = 1435.8 ft MSL (50-year elevation); 2% (50-year) flood hydrograph.

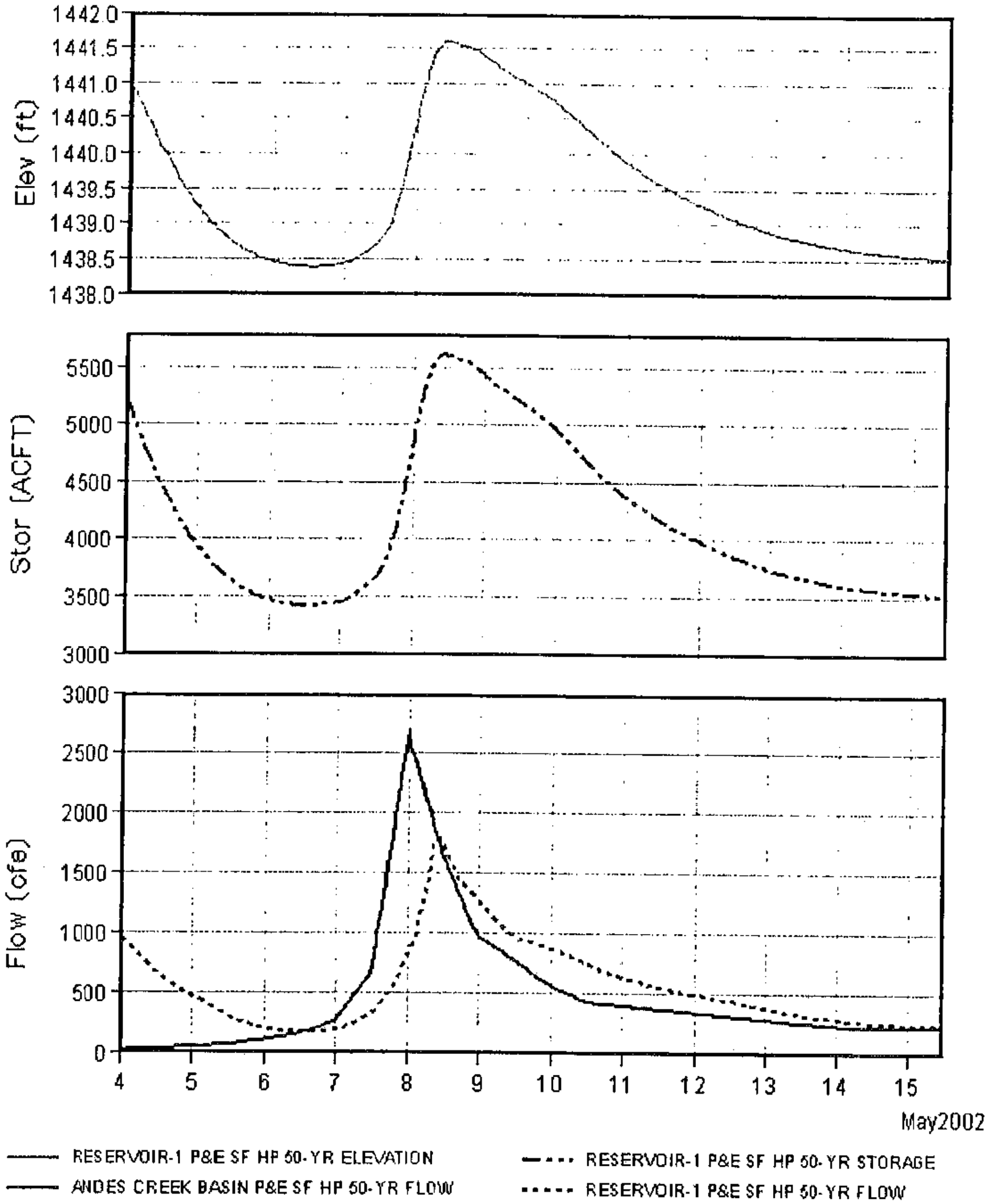

Figure E.5 Lake Andes North Unit HEC-HMS flood routing analysis: Starting pool elevation = 1441.0 ft MSL (50-year elevation); 2% (50-year) flood hydrograph.

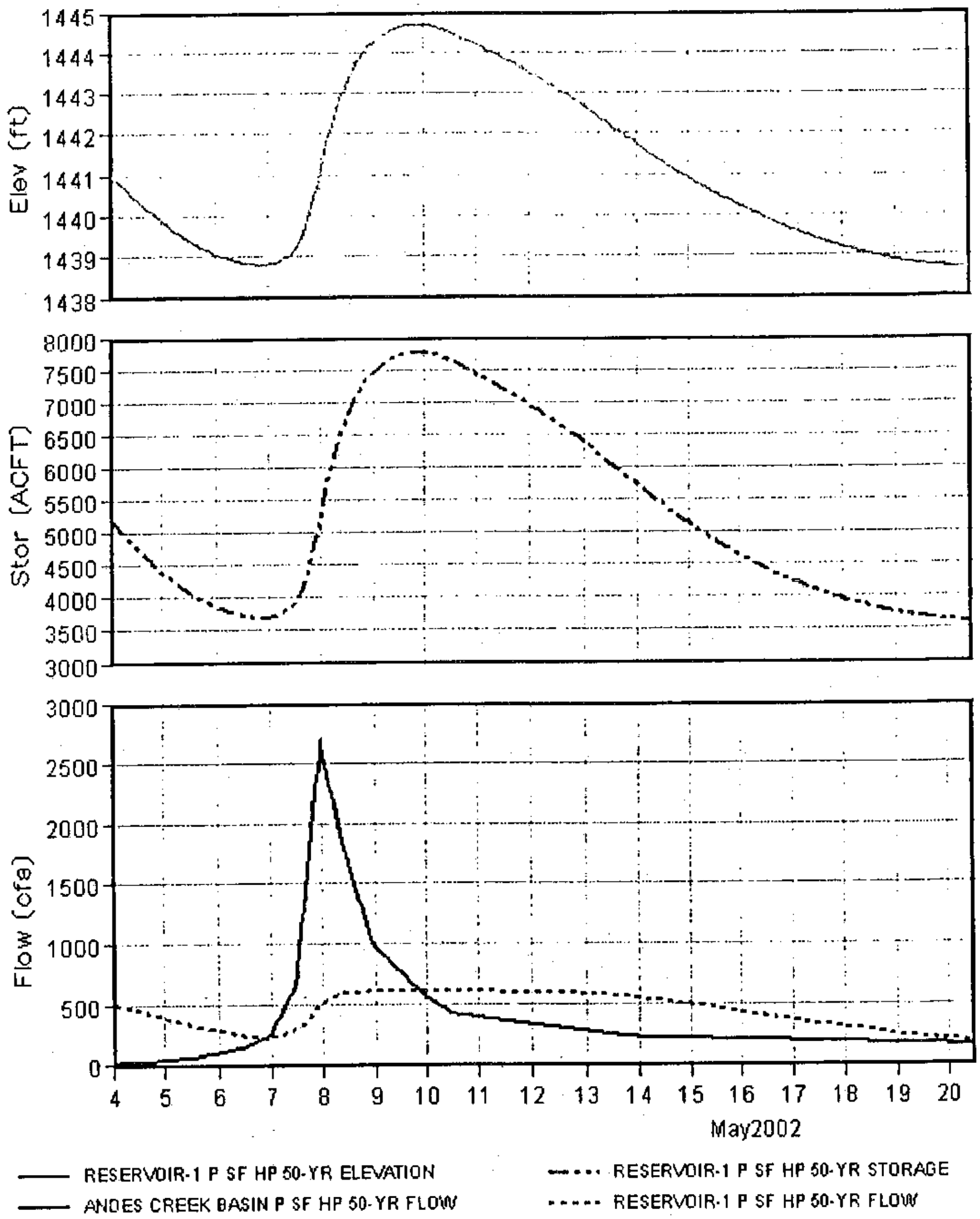

Figure E.6 Lake Andes North Unit HEC-HMS flood routing analysis with tailwater reduction of outflow (US 281 controlled): Starting pool elevation = 1441.0 ft MSL (50-year elevation); 2% (50-year) flood hydrograph.